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The Association
of
Engineering and Shipbuilding
Draughtsmen.

NOTES ON THE DESIGN OF MILD STEEL CHIMNEYS.

By D. L. HENSHAW, B.A., Hons. (London).

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Note. Ratio. Height : Diameter.

is approx. 20 : 1.

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R. H. MANN.

NOTES ON THE DESIGN OF MILD STEEL CHIMNEYS.

By D. L. HENSHAW, B.A., Hons. (London)

INTRODUCTION.

There are three types of chimney in general use—brick, reinforced concrete, and steel. All have certain features in common. It is usual for the height and diameter to be specified by the purchaser, since natural draught cannot always satisfy the needs of modern plant. Furthermore, local bye-laws frequently fix the minimum height of stacks, that for chemical works, for instance, being in the region of 250′ 0″.

From a functional point of view all chimneys have to resist the effects of the wind and of their own weight. In the case of brick chimneys the self-weight is usually the more important, while with steel chimneys, which are a fraction of the weight of a brick stack of the same height and diameter, the effects of the wind predominate. Reinforced concrete stacks lie approximately mid-way between these two extremes. With respect to foundations, which are the last link between chimney and earth, all types of chimney rely on weight to oppose wind effects.

Brick chimneys are built with a circle or any regular polygon as their cross section, square, hexagon, or octagon being common shapes, and they are frequently topped by an enlarged cap to assist in the emission of smoke. Steel and concrete chimneys are usually circular in section, this shape being the most efficient.

Steel chimneys, by virtue of their relatively low cost and weight, speed of erection and demolition, and immunity from damage by lightning when earthed, have many advantages over their brick and concrete counterparts. Also they suffer much less from the effects of temperature stresses, because their walls are thinner and conduct heat far better.

Admittedly the life of a steel chimney—30 years or so—is less than that of a brick or concrete one, but generally this is not a serious matter, since it is normally long enough for most plants in these days of rapidly changing processes. Brick chimneys usually outlive their use and then have to be demolished at no little expense.

Classes of Steel Chimneys.

Steel chimneys, with which this pamphlet deals, can be classified according to principle of design, size, and method of construction.

TYPE 1. The self-supporting type acts as a column resisting its own weight, acting vertically, and the overturning effects of the wind, the base only being fixed. In other words it is a cantilever subjected to a relatively large overturning moment and a relatively small vertical axial load. In this type of chimney the lower portion is usually coned out to some one-and-a-half-times the diameter of the upper portion, the coned portion being about 1/5th of the total height.

TYPE 2. The guyed chimney resists the same forces, but it is assisted by lateral supports near the top, these generally being in the form of three or four wire ropes equally spaced around the chimney and anchored at the other end to heavy concrete blocks some distance from the chimney footings. The effect of these guys is to produce a somewhat smaller wind bending moment in the chimney, which now acts as a beam with two reactions.

Normally the self-supporting chimney is preferred, because it occupies less ground space—a matter of importance in congested areas—but guys may be necessary where inadequate foundations exist or when it is desired to increase the height of an already constructed self-supporting chimney.

The size of a chimney will determine, for transport reasons, whether it is despatched in separate plates to be assembled on site or whether it can be largely shop assembled. The larger chimneys are in any case built up on site because of the great amount of shop space they would otherwise require.

Site erected chimneys are usually of riveted construction with lap joints, the upper section overlapping the lower to make a weathertight joint. Welded chimneys are sometimes met with, it being claimed that the butt joints possible with welding are less liable to corrosion than the lap joints of the riveted type. The main objection to welding is the difficulty of ensuring good fitting at the seams and sound welds under frequently adverse conditions.

The smaller chimneys can more easily be built up into fairly long sections before leaving the shops, and for these welding is preferred, especially with sizes which do not allow easy access inside. Site joints are usually effected by bolting external flanges together, although lap joints with bolts or rivets are possible.

Materials Used.

Steel chimneys are built as a rule of structural or tank quality mild steel plates (the minimum thickness being $\frac{1}{4}''$ except in the case of very small stacks) and structural quality mild steel sections, rivets, bolts, etc. Sometimes copper bearing steel or "Armco" iron is employed. The external surfaces are usually protected by heat resisting bituminous paint applied after flame descaling or when mill scale, etc., have been removed by several months of exposure to the elements. Occasionally coatings of concrete are specified.

Internal surfaces may be covered with heat resisting paint, but more frequently the plates are protected from the effects of hot and corrosive gases by firebrick, concrete, or asbestos linings, especially in the lower portions. Firebricks are supported on internal steel rings spaced at intervals down the chimney, while concrete (or "gunite," as they are frequently called) and asbestos linings are formed by a special process of spraying on to a steel fabric cleated or spot welded to the chimney plates.

The circular steel sections are despatched in one or more plates per section according to the diameter of the chimney, and a convenient height of these sections is such as to allow 5′ 0″ centres of horizontal seams. The circular sections are usually called "strakes."

Footings are normally made from concrete with mesh reinforcement, such footings to be strong enough to withstand the compressive stress from the steel chimney, large enough to limit the compressive stress on the earth to that allowed for the ground in question, and heavy enough to prevent overturning of the chimney.

Loads.

The loading imposed by the wind is allowed for in two ways. In many instances the client specifies that "the chimney shall be designed to withstand a wind force of so many pounds per square foot acting on the projected area." The force so specified varies between 20 and 30 pounds per square foot. The term "projected area" means that area having a width equal to the diameter of the chimney and a height equal to the height of the chimney exposed to the wind. This area should also include the trapezium shape formed by the coned out portion of the stack if exposed.

Alternatively the wind force may be calculated from accepted formulae, one of which, namely that given in B.S.S. 449, 1948, makes allowance for height, geographical location, degree of exposure, and circularity. This formula calls for the footings and immediate base fixings to be suitable for withstanding 150% of the wind load used in designing the stack itself, and it is advisable in the case of chimneys with linings or coatings, to check stability for two conditions, namely 100% wind force, neglecting the weight of the lining or coating, and 150% wind force allowing for the weight of the lining or coating. When chimneys are for destinations where high velocity winds occur it is wise to design for wind forces somewhat in excess of those allowed for in this country.

The other load to take into consideration is that imposed by the weight of the steel chimney and fittings, and by any coating or lining. In the case of guyed chimneys, the vertical component of the guy pull must be taken into account. The effects of deflection due to wind and of misalignment during erection can usually be ignored.

With respect to the steel weight, it is sufficiently accurate to calculate the nominal plate area, *i.e.*, circumference times height, in square feet, and multiply this by the weight per square foot of such plate. To this should be added some 20 to 30 per cent. to cover for laps, rivets, fittings, etc. A value of 140 lbs. per cubic foot is suitable for firebrick or concrete linings, etc., An easy way of determining weights for linings is to work on the area of the annular cross section of the lining (obtained from tables) and multiply this by the height and density, units being pounds and feet.

When designing the base of the chimney it is necessary to make some further estimated addition to the chimney weight as calculated above to allow for the base plate, etc., and in the case of a final design it is advisable to use weights accurately worked out from the actual chimney drawing. It should be remembered, however, that vertical loads for steel chimneys are generally of less importance than wind loads.

It is usually specified that the firebrick lining in the parallel portion only of the chimney is to be carried by the steel shell, so that the lining in the coned out portion of the chimney rests directly on the concrete footing and does not affect the steel design. Nevertheless, the weight of this latter portion of the lining, together with any earth, etc., placed on top of the concrete footing outside the chimney, can be taken into consideration when the footing is designed.

Working Stresses.

The maximum compressive stress—derived by adding the stress due to wind forces to that resulting from vertical loads—generally does not exceed three tons per square inch for chimney plates; and it is wise to reduce this in the case of large diameter stacks, since the stiffening effect of the curvature is not so pronounced. As a rule the tensile stress in the plates—derived by subtracting weight stress from wind stress—need not be calculated. It is not possible to be very definite about plate stresses, because they usually depend on considerations of economic riveting.

For the actual calculation of plate stresses, the stress due to wind forces is found by dividing the bending moment in inch tons by the modulus of section of the chimney at the point in question. *The modulus of section, or Z, is approximated to $\pi r^2 t$ inch units, where r = internal radius in inches and t = plate thickness in inches. Tables for the areas of circles are useful in this calculation, since they give πr^2 values.

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* See Appendix 2, Part (*a*).

The stress due to vertical loads can be determined by dividing the total load (as detailed earlier) expressed in tons, by the cross sectional area of the chimney at the point in question. It is frequently easier to work out the load imposed by the twelve inch wide strip of plate, etc., above the point in question and divide this by the cross sectional area of the strip. When the tensile stress in the plate is calculated, due attention should be paid to the efficiency of the plate along a section through the rivet holes.

For rivets the maximum shear stress should not exceed 5 tons per square inch, while the maximum bearing stress permitted is usually 10 tons per square inch. Some specifications limit this latter stress to 7·5 tons per square inch. The tensile stress in foundation bolts, working on the area at the bottom of the thread, generally does not exceed 5 tons per square inch, while the tensile stress in rivets can be taken at 4 tons per square inch.

Suitable stresses for the preliminary design of concrete footings are :—steel to concrete, 15 - 20 tons per square foot ; concrete to earth, 2 tons per square foot. These stresses should be checked when fuller information is available. It is probably wisest to be conservative where footings are involved in order to limit the deflection of the steel chimney under the effects of the wind.

With respect to guy ropes it is best to consult reputable makers, who will be pleased to advise and quote if details of the design load and site conditions are submitted to them. A factor of safety of 6 is usually allowed.

Riveting.

Rivets used in chimney construction range between $\frac{5}{8}''$ diameter for the lightest chimney to 1″ for the lower portions of the larger chimneys. $\frac{3}{4}''$ diameter is a very useful size, since such rivets can readily be driven by pneumatic hand tools and are generally suitable for this class of work. To produce joints of sufficient strength in the coned-out section of stacks it is sometimes advisable to use $\frac{7}{8}''$ or even 1″ diameter rivets, but it is usually better to adhere to one size of rivet throughout the entire chimney.

The following are recommended minimum distances for rivets :

Nominal diameter of rivet	$\frac{5}{8}''$	$\frac{3}{4}''$	$\frac{7}{8}''$	1″
Minimum pitch of rivet ...	2″	$2\frac{1}{2}''$	3″	$3\frac{1}{2}''$
Minimum edge distance ...	$1\frac{1}{8}''$	$1\frac{1}{4}''$	$1\frac{1}{2}''$	$1\frac{3}{4}''$
Minimum centres of rows ... (rivets staggered)	$1\frac{1}{2}''$	2″	$2\frac{1}{4}''$	$2\frac{5}{8}''$

The maximum pitch of rivets rarely exceeds 5″ even when several rows of rivets are used at a seam, since it is necessary to avoid undue leakage at joints. A clearance of $\frac{1}{16}''$ is usually allowed in drilling rivet holes in order to assist in assembly, and the diameter of the hole is taken as the effective size of the driven rivet for the purposes of calculating stresses.

Where more than one row of rivets are used the rivets are staggered, the pitch generally remaining constant. In some of the remarks that follow the term "equivalent pitch" is used to denote the pitch between rivets divided by the number of rows. Thus a treble riveted seam with rivets at $4\frac{1}{2}''$ pitch would have an equivalent pitch of $1\frac{1}{2}''$.

The plate thickness, pitch of rivets, number of rows, and size of rivets are interconnected, and a choice of plate thickness cannot be made until all points involved are given due consideration. It is hoped that Tables 1 to 4 will assist in this respect, although it is frequently necessary to make calculations before reaching a conclusion. In any case it is best to keep well within the stress limits prescribed. This applies also to vertical seams, where exact calculation is of doubtful value.

Mathematically the various points relating to riveting can be expressed by the equations below, where

D = diameter of driven rivet, in inches.

p = equivalent rivet pitch, in inches.

P = centre of rivets in the outer row of a multiple seam, in inches (equals p in a single riveted seam).

f_s and f_b = shear and bearing stresses respectively in the rivets, in tons per square inch.

f_c and f_t = compressive and tensile stresses respectively in the chimney plate, in tons per square inch.

t = thickness of chimney plate, in inches.

$$f_c \times p \times t = f_s \times \pi D^2/4 \qquad \text{equation (1) (shear)}$$

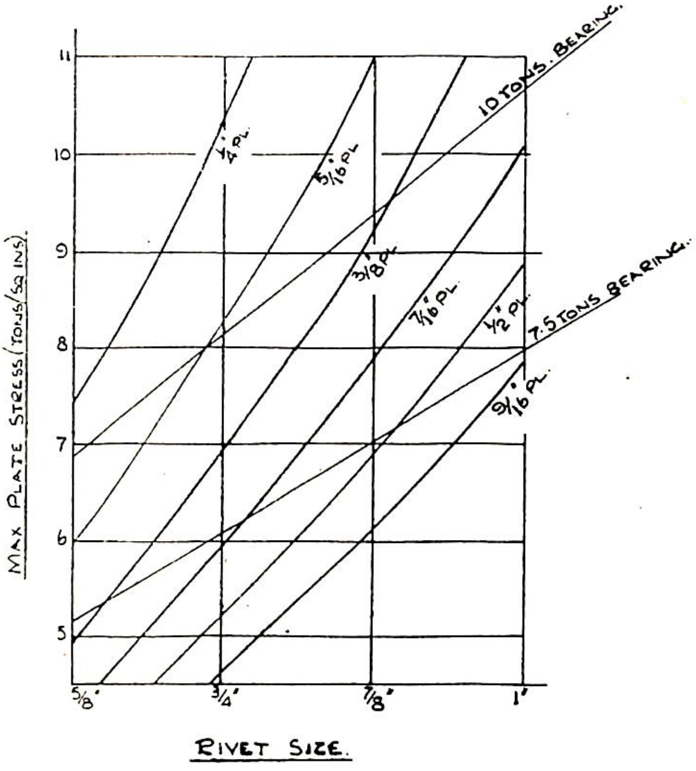
$$f_c \times p \times t = f_b \times D \times t$$

$$\text{or } f_c \times p = f_b \times D \qquad \text{equation (2) (bearing)}$$

$$\text{Tensile stress (allowing for drilling) in plate} = f_t \times \frac{P}{P - D} \qquad \text{equation (3)}$$

Equations similar to (1) and (2) can be made involving f_t but they are unnecessary, since the compressive stress in the chimney plate, being the larger, determines the stresses in the rivets as far as maximum values are concerned.

Figure 1 sets out the maximum values of f_c as derived from equations (1) and (2), where $p = 1''$, and f_s and f_b = 5 and 10 (alternatively 7·5) tons per square inch respectively, and shows for all pitches whether shear or bearing in the rivets shall be taken as the criterion for deciding on rivet pitch, etc. For instance, with $\frac{3}{4}''$ dia. rivets and 10 tons per square inch bearing, bearing is the criterion for plate thickness up to $\frac{5}{16}''$, and shear for greater thicknesses.



CURVED LINES FOR 5 TONS/SQ.INS. SHEAR

STRAIGHT LINES FOR BEARING

EQUIVALENT RIVET PITCH 1"

FIG 1

Table 1 illustrates equation (1) for plates between $\frac{3}{16}''$ and $\frac{3}{4}''$ thick, and for rivets between $\frac{5}{8}''$ and $1''$ diameter before driving. It is intended for use with a transparent setsquare, one 90° edge being placed across the lines marked plate thickness and equivalent rivet pitch, while the other 90° edge lies across the lines marked plate stress and shear stress in rivets (4 sets of graduations are provided for the different rivet sizes). Thus a typical reading—shown by the dotted lines—gives an equivalent pitch of $3\frac{1}{2}''$ when f_c, f_s, and t are respectively 2·36, 5, and $\frac{5}{16}''$, and the rivets are $\frac{3}{4}''$ size.

Table 2 is similar to Table 1 but applies to thicker plates.

Table 3 is a corresponding illustration of equation (2), which is independent of plate thickness.

Table 4, which is to be read in the same way as Table 1, shows the relation between the tensile stresses in the undrilled and the drilled plate, *i.e.*, along a section through the outer row of a riveted seam, for various pitches and rivet sizes. As a rule riveted seams in chimney construction are efficient enough to keep this nett tensile stress within safe limits.

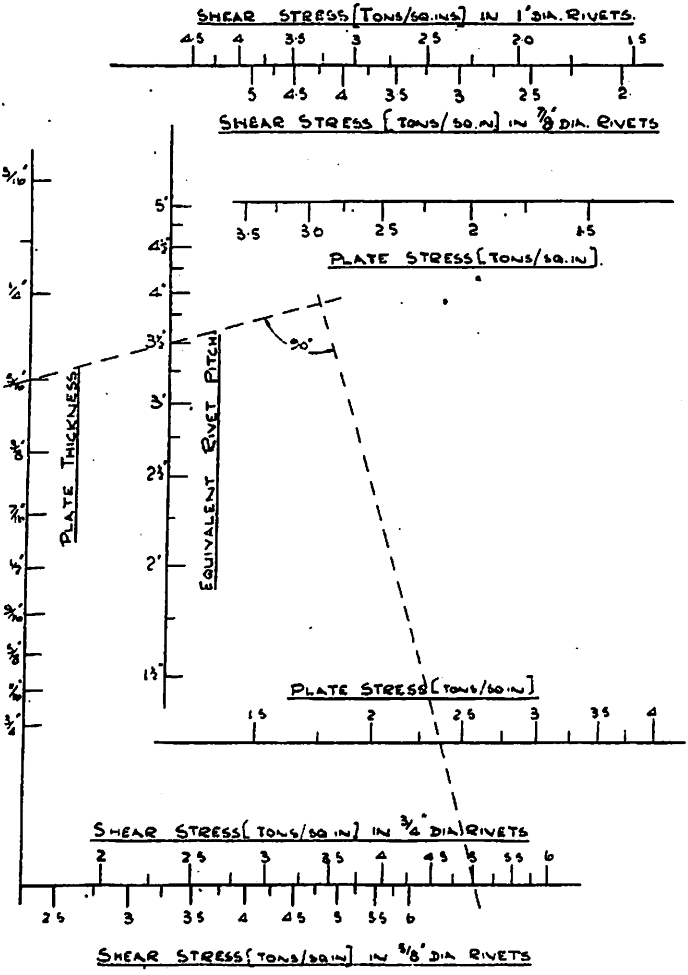


TABLE 1 (SHEAR)

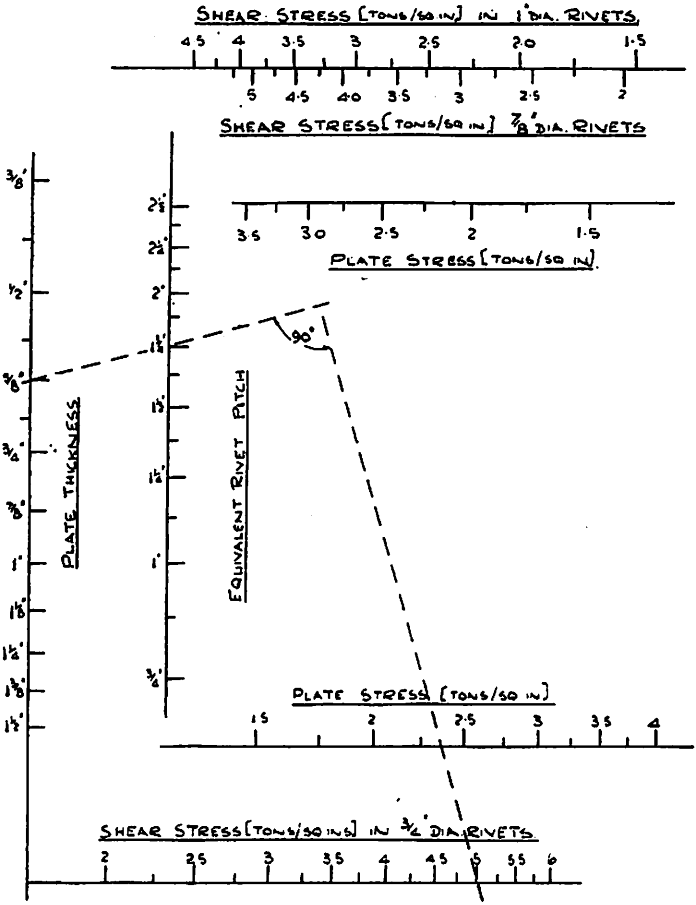


TABLE 2. (SHEAR)

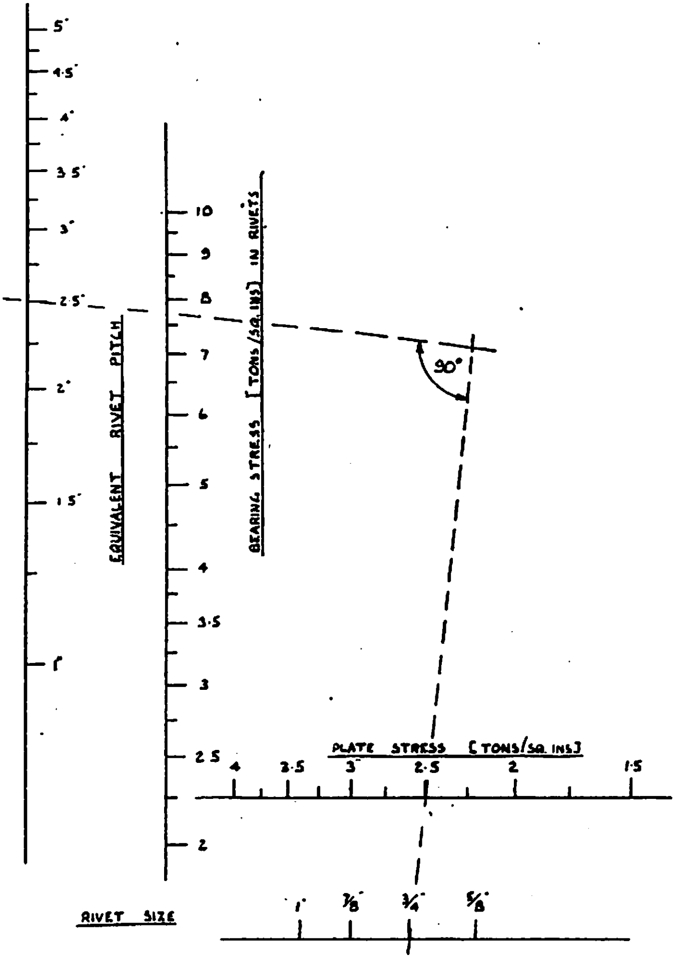


TABLE 3 [BEARING]

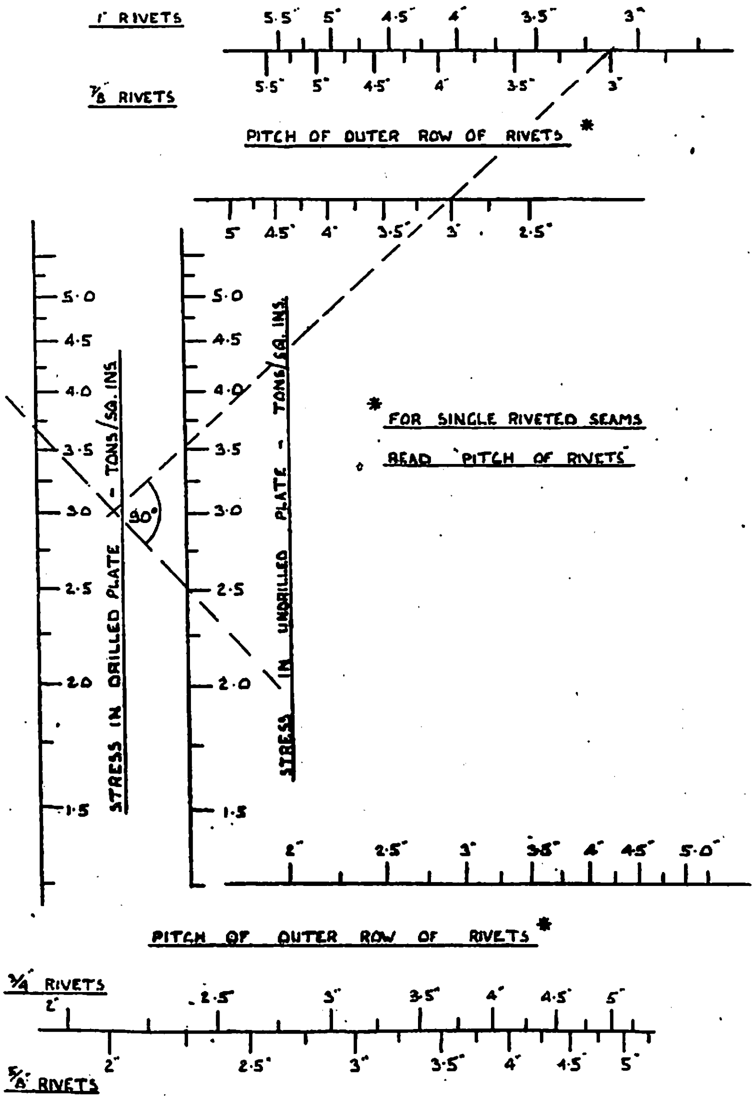


TABLE 4 [TENSION IN PLATE]

EXAMPLES OF DESIGN.

(1) Self-Supporting Unlined Chimney 7′ 0″ I/Dia. × 140′ 0″ High.

Client's Specification.—Located in N.W. England ; conditions of maximum exposure ; wind to B.S.S. 449, 1948 ; full height exposed to wind. Base coned out to 10′ 6″ i/dia. over lower 30′ 0″ ; riveted design. Flue inlet 6′ 0″ × 3′ 6″ at 15′ 0″ from base. Maximum stresses in rivets—5 tons per square inch shear, 7·5 tons per square inch bearing. Soil suitable for maximum stress of 2 tons per square foot. (See Figs. 2 and 2A).

Reading from Tables 3 and 4 in B.S.S. 449, $v = 80$ and $h = 140$, therefore $p = 32{\cdot}7$ lbs. per square foot. Allowing for the rotundity factor of 0·7, the wind force becomes $32{\cdot}7 \times 0{\cdot}7 = 22.89$ or, say, 23 lbs. per square foot.

Using the notation :—

W = weight of chimney above any given point, in tons.

A = cross section of chimney at any given point, in square inches.

M = bending moment in chimney at any given point due to wind, in inch tons.

*Z = modulus of section of chimney at any given point, in inch units, and approximated to π × square of i/radius × thickness of plate.

f = stress in chimney plate at any given point resulting from the effects of self weight and wind, in tons per square inch.

h = height down to any given point from the top of the chimney, in feet.

and assuming the upper plates to be $\frac{1}{4}''$ thick and weighing (to allow for laps, fittings, etc.) 13 lbs. per square foot, then, for the parallel portion

$$W = \frac{13 \times \pi \times 7 \times h}{2240} \qquad \text{and } A = \frac{\pi \times 7 \times 12}{4}$$

$$\therefore \frac{W}{A} = \frac{13 \times \pi \times 7 \times h}{2240} \times \frac{4}{\pi \times 7 \times 12} = \frac{13 \times h}{2240 \times 3}$$

$$= 0{\cdot}00193h \qquad (1)$$

It should be noted that this expression is independent of the plate thickness when this is constant for the full height h, but must be modified in the ratio of the thinner to thicker plate thickness to give the stress imposed on thicker plate by a height h of thinner plate. Thus if we have h_1 in $\frac{1}{4}''$ plate followed by h_2 in $\frac{3}{8}''$ plate, the total W/A stress in the $\frac{3}{8}''$ plate is $0{\cdot}00193\,(h_1 \times \frac{2}{3} + h_2)$. Also the wind load—for the parallel portion only—on the projected area is $\frac{23 \times 7 \times h}{2240}$ tons acting at $\frac{h \times 12}{2}$ inches above the given point in question.

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* See Appendix 2, Part (*a*).

$$\text{Hence } M = \frac{23 \times 7 \times h^2 \times 12}{2 \times 2240} \quad \text{also } Z = \frac{\pi \times 3{\cdot}5^2 \times 12^2}{4}$$

$$\therefore \quad \frac{M}{Z} = \frac{23 \times 7 \times h^2 \times 12}{2 \times 2240} \times \frac{4}{\pi \times 3{\cdot}5^2 \times 12^2}$$

$$= \frac{23 \times h^2}{2240 \times 3 \times 3{\cdot}5 \times \pi} = 0{\cdot}000312h^2 \qquad (2)$$

[This second expression is for $\frac{1}{4}''$ thick plate, and if, say, $\frac{5}{16}''$ plate is used it should be reduced by $\frac{1}{5}$ (the ratio $\frac{1}{4}$ to $\frac{5}{16}$)].

Consulting Fig. 1 it is seen that, assuming $\frac{3}{4}''$ rivets to be used, bearing is the criterion for plates up to $\frac{3}{8}''$ thick with a maximum rivet bearing stress of 7·5 tons. Referring then to Table 3, it is found that the maximum plate stress to permit single riveting at 3″ pitch is 2 tons per square inch. 3″ pitch is chosen as a reasonable pitch to maintain as far down the chimney as possible.

Equating the sum of (1) and (2) above to 2 tons per square inch $0{\cdot}00193h + 0{\cdot}000312h^2 = 2$, or $h^2 + 6{\cdot}2h - 6400 = 0$, whence $h = 77$ or, say, 75′ 0″. Therefore $\frac{1}{4}''$ plate with single rivets at 3″ pitch can be used for the first 75′ 0″.

It will be assumed now that $\frac{5}{16}''$ plate with the same pitch is used for a further distance of h_1 feet down the stack. Then

$0{\cdot}00193\ (75 \times 4/5 + h_1) + 0{\cdot}000312\ (h_1 + 75)^2 \times 4/5 = 2$

or $h_1^2 + 150h_1 + 5{,}625 + 7{\cdot}75\ (60 + h_1) - 8{,}000 = 0$.

or $h_1^2 + 157{\cdot}75h_1 - 1{,}910 = 0$, whence $h_1 = 11{\cdot}3$ or, say, 10′ 0″. Therefore $\frac{5}{16}''$ plate with rivets as above is suitable for the next 10′ 0″ down.

Assuming now that double rivets will be used for the remainder of the parallel portion, and that $\frac{5}{16}''$ plate with a maximum stress of 3 tons per square inch is used for a further distance of h_2 feet down the stack, then

$0{\cdot}00193\ (75 \times 4/5 + 10 + h_2) + 0{\cdot}000132\ (h_2 + 85)^2 \times 4/5 = 3$

or $h_2^2 + 170h_2 + 7{,}225 + 7{\cdot}75\ (70 + h_2) - 12{,}000 = 0$

or $h_2^2 + 177{\cdot}75h_2 - 4232{\cdot}5 = 0$, whence $h_2 = 21{\cdot}25$ or, say, 20′ 0″. Therefore $\frac{5}{16}''$ thick plate with a maximum stress of 3 tons per square inch is suitable for a further distance of 20′ 0″ down, *i.e.*, to a total of 105′ 0″ down.

It is assumed that the remainder of the stack will be $\frac{3}{8}''$ thick, and stresses will be calculated for the following distances from the chimney top :—75′ 0″, 85′ 0″, 105′ 0″, 110′ 0″ (where the cone begins), 125′ 0″ (where the flue inlet is required), and 140′ 0″.

For the parallel portion, the formulae and equations previously used can be employed.

Thus, tabulating :—

Height Down	$\frac{W}{A}$ Stress	
75′ 0″	$0{\cdot}00193 \times 75$	$= 0{\cdot}145$ tons/sq. ins.
85′ 0″	$0{\cdot}00193 \times (75 \times \frac{4}{5} + 10)$	$= 0{\cdot}135$,,
105′ 0″	$0{\cdot}00193 \times (75 \times \frac{4}{5} + 30)$	$= 0{\cdot}174$,,
110′ 0″	$0{\cdot}00193 \ (75 \times \frac{2}{3} + 30 \times \frac{5}{6} + 5)$	$= 0{\cdot}154$,,
	$\frac{M}{Z}$ Stress	
75′ 0″	$0{\cdot}000312 \times 75^2$	$= 1{\cdot}76$,,
85′ 0″	$0{\cdot}000312 \times 85^2 \times \frac{4}{5}$	$= 1{\cdot}81$,,
105′ 0″	$0{\cdot}000312 \times 105^2 \times \frac{4}{5}$	$= 2{\cdot}76$,,
110′ 0″	$0{\cdot}000312 \times 110^2 \times \frac{2}{3}$	$= 2{\cdot}52$,,

With respect to the cone, the weight of the parallel portion, taking ¼″ plate to weigh 13 lbs. per square foot and other thicknesses in proportion, is

$$\frac{13 \times 7 \times \pi}{2240} (75 + 30 \times 5/4 + 5 \times 6/4) = 15{\cdot}3 \text{ tons.}$$

The average diameter of the cone is $\frac{7 + 10{\cdot}5}{2} = 8{\cdot}75'$, therefore, assuming a slant height of 31′ 0″, the approximate weight of the whole cone is

$$\frac{8{\cdot}75 \times \pi \times 31 \times 13 \times 6}{2240 \times 4} = 7{\cdot}4 \text{ tons.}$$

Similarly the weight of the cone down to the 125′ 0″ level is

$$\frac{15{\cdot}75 \times \pi \times 15{\cdot}5 \times 13 \times 6}{2 \times 2240 \times 4} = 3{\cdot}4 \text{ tons.}$$

Therefore, since the respective cross sectional areas at 125′ 0″ and 140′ 0″ down are $8{\cdot}75 \times \pi \times 12 \times \frac{3}{8}$, and $10{\cdot}5 \times \pi \times 12 \times \frac{3}{8}$, or 124 and 149 square inches, the respective W/A stresses are $\frac{15 \cdot 3 + 3 \cdot 4}{124}$

and $\frac{15 \cdot 3 + 7 \cdot 4}{149}$ or 0·151 and 0·152 tons per square inch.

The bending moment at the base level is total wind load × leverage, and is best found by adding the results for the portion 7′ 0″ × 140′ 0″ and for the two triangular portions each 1′ 9″ wide at the base × 30′ 0″ high which make up the cone. Thus

$$\frac{23 \times 7 \times 140}{2240} \times \frac{140}{2} \times 12 = 0 \cdot 43 \times 140^2 = 8425 \text{ inch tons}$$

$$\frac{23 \times 1 \cdot 75 \times 30}{2240} \times \frac{30}{3} \times 12 = 0 \cdot 072 \times 30^2 = 65 \text{ ,, ,,}$$

$$\text{Total} = \underline{\underline{8490}} \text{ ,, ,,}$$

Likewise at 125′ 0″ down the bending moment is

$$0 \cdot 43 \times 125^2 + 65 \times \left(\frac{15}{30}\right)^3 = 6700 + 8 = 6708 \text{ inch tons.}$$

The values of Z at the two levels being $\frac{\pi \times 63^2 \times 3}{8}$ and

$$\frac{\pi \times 52 \cdot 5^2 \times 3}{8},$$

or 4660 and 3250 inch units, the respective M/Z stresses are

$$\frac{8490}{4660} = 1 \cdot 82, \text{ and } \frac{6708}{3250} = 2 \cdot 07 \text{ tons per square inch.}$$

The above values are tabulated below, together with approximate rivet stresses and tensile plate stresses, allowing for rivet holes. The riveting taken is, upper 85′ 0″, ¾″ rivets, 3″ single pitch, 2½″ lap; remainder ¾″ rivets, 4″ double pitch, 4½″ lap. The vertical seams are assumed to be ¾″ rivets, 3″ single pitch, 2½″ lap throughout, and here no calculations are required. This is due to the fact that theoretical calculations give results which are quite impractical in most cases for small diameters.

(See Appendix 2 for further information on this point).

For the parallel portion 2 plates per strake will suffice, while in the conical portion three plates per strake are advisable.

Location	$\frac{M}{Z}+\frac{W}{A}$	Rivet Bearing	Rivet Shear	$\frac{M}{Z}-\frac{W}{A}$	Tensile Plate Stress	t
75′ 0″ down	1·905	7·25	2·85	1·615	2·25	$\frac{1}{4}''$
85′ 0″ ,,	1·945	7·30	3·60	1·675	2·30	$\frac{5}{16}''$
105′ 0″ ,,	2·934	7·25	3·60	2·586	3·25	$\frac{5}{16}''$
110′ 0″ ,,	2·674	6·75	3·80	2·366	3·00	$\frac{3}{8}''$
125′ 0″ ,,	2·221	5·60	3·25	1·919	2·50	$\frac{3}{8}''$
140′ 0″ ,,	1·972	5·00	2·85	1·668	2·15	$\frac{3}{8}''$

(The stresses given are read from Tables 1 - 4 and are approximate only).

Flue Inlet.

Where the 3′ 6″ wide opening is cut for the flue inlet, some form of compensation is required to take the loading. Joist or channel section is usually employed, and it is advisable to fit the compensation outside the stack where it can be inspected and where it is less likely to corrode. While the flue inlet, being built up from steel plates, assists in stiffening the chimney, calculations frequently ignore this fact because of corrosion effects. There are several methods of arriving at a suitable section for the compensation members, all giving comparable results. In any case the section must be suitable as a strut, and should be extended above and below the flue inlet to give space for sufficient rivets, and to allow the loading concentrated in the compensation section to be dispersed.

In the calculations below an endeavour is made to keep the neutral axis through the compensated section on the centre line of the whole chimney in order to avoid eccentric loading and a change in direction of stresses, but it is not easy to obviate some eccentric loading on the compensating members themselves, together with some locally higher stresses on adjacent plate. Such eccentric loading is offset in part by the fact that the line of action of the loading in the chimney plates above and below the flue section is often as far from the centre line of the chimney as the neutral axis of the compensating members is.

Assuming an average diameter of 8′ 9″ at the flue section, the first moment of area of the plate removed is, where 2θ is the angle in radians subtended at the chimney centre by the arc removed, and r the radius,

$$\int_{-\theta}^{\theta} r^2 \cos \theta d\theta \times \tfrac{3}{8}'' = 52{\cdot}5^2 \times \sin \theta \times 2 \times \tfrac{3}{8} \text{ inch units.}$$

But $\sin\theta = \dfrac{21}{52{\cdot}5}$, therefore the value becomes $52{\cdot}5 \times 21 \times 2 \times \frac{3}{8}''$.

Now the perpendicular distance from the point of intersection of the flue plate with the chimney plate to a chimney centre line at right angles to the flue centre line is $\sqrt{52{\cdot}5^2 - 21^2}$ or 48″. Assuming a distance of 53″ to the centre of the compensating sections to allow for the depth of the sections, and letting each section have an area A, then, if two such sections are provided,

$2\text{A} \times 53 = 52{\cdot}5 \times 21 \times 2 \times \frac{3}{8}''$ or $\text{A} = 7{\cdot}80$ sq. inches.

1 R.S.C. 11″ × 3½″ × 26·78 lbs. (area 7·88) per side will be suitable.

As a rough check the approximate load imposed on each stiffener is, using previous figures,

$$2{\cdot}221 \times 124 \times \frac{\theta}{2\pi} = 2{\cdot}221 \times 124 \times \frac{24^\circ}{360^\circ} = 18 \text{ tons say, and}$$

the axial compression is $\dfrac{18}{7{\cdot}88} = 2{\cdot}3$ tons per square inch. The permitted stress, assuming an effective strut length of 72″, is 5·5 tons per square inch, *i.e.*, 42% is used. Allowing also for an eccentric bending moment of 18 × 5½, or 99 ins. tons, the additional percentage stress is $\dfrac{99}{25{\cdot}8} \times \dfrac{100}{10}$ or, say, 38%. Thus the selected stiffener is adequate, since 80% only of the total strength is used.

To determine the number of rivets required at the top and bottom, the load to be taken is approximately 18 tons, and 1 – ¾″ dia. rivet in bearing can take a load of $7{\cdot}5 \times \frac{13}{16} \times \frac{3}{8} = 2{\cdot}3$ tons (7·5 tons stress, ⅜″ plate). Thus 18 ÷ 2·3 or 8 rivets are required and these can be accommodated in a length of 2′ 0″ of channel flange at 3″ pitch.

If it is desired to check the effect of the compensated opening for the flue on the strength of the chimney, the Z of which is 3250 inch units at the level of the centre line of the flue before any opening is cut (see earlier), this can be achieved thus, working about a chimney centre line perpendicular to the flue.

The second moment of area of the ⅜″ thick arc removed is approximately

$$\int_{-\theta}^{\theta} r^3 \cos^2\theta \, d\,\theta \times t = \int_{0}^{\theta} r^3 (1 + \cos 2\theta)\, d\,\theta \times t = r^3\left(\theta + \frac{\sin 2\theta}{2}\right) \times t$$

Now $\theta = 24^\circ$ approximately $= 0{\cdot}42$ radians, and $\dfrac{\sin 2\theta}{2} = \dfrac{\sin 48^\circ}{2}$

$= 0{\cdot}3716.$

Therefore the reduction for the arc removed is

$$52{\cdot}5^3 \times \tfrac{3}{8} \times (0{\cdot}42 + 0{\cdot}3716) = 42{,}700 \text{ inch units.}$$

The approximate second moment of area of the channel sections, taking 53" to the neutral axis from the chimney centre line, is

$$2 \times 7{\cdot}88 \times 53{\cdot}0^2 = 44{,}200, \text{ or an increase of 1500 inch units.}$$

But the second moment of area of the original section is $3250 \times 52{\cdot}5 = 170{,}000$, therefore the compensated value is 171,500 inch units. Now the extreme corner of the channel is some 58" from the chimney centre line, giving a Z of $171{,}500 \div 58 = 2950$ inch units, or 91% of the original value. It should be remembered, however, that the bending moment divided by this value of Z gives the stress in the extreme fibres of the channel stiffeners, whereas the stress in the chimney plates themselves is actually lessened in the ratio $\dfrac{170{,}000}{171{,}500}$, *i.e.*, original second moment of area divided by new second moment of area. Thus the chimney Z is not seriously weakened, and the plate stress is reduced.

It will be clear on inspection that no check about the centre line of the chimney parallel to the flue is necessary.

Base.

Calculations to determine the number and size of the holding down bolts for the chimney, and the thickness and width of the base plate are to some extent based on empirical methods, and there are several ways of arriving at comparable results. This is due to the fact that chimney bases are rarely rigid enough to justify the assumption that stresses vary as the first power of the distance from a neutral axis, or that the relation between the moduli of elasticity of steel and concrete should be considered.

In the method which follows it is assumed that only a certain leeward arc of the chimney base plate, which is usually in the form of two concentric circles or octagons, exerts a pressure on the concrete footing when the wind tends to overturn the chimney. The assumed arc is generally about 90°, and the pressure so exerted is considered to be concentrated at a point called the centre of pressure, or C.P., which is determined by inspection of the proposed base layout. The moment about the C.P. resulting from the pull of the holding down bolts taken to be effective and the weight of the chimney, acting centrally, is then equated to the overturning moment of the wind, and the size of bolt obtained. The holding down bolt pull is added to the weight of the chimney to give the total pressure exerted by the arc mentioned above on the concrete footing, and the base plate is made wide enough to keep the stress on the concrete within the prescribed limit, and strong enough to prevent undue deflection.

The stress in the holding down bolts, which usually number 8 or 16, is considered to be constant irrespective of their distances from the C.P., those between the centre of the base and this point being neglected. Likewise the stress on the concrete footing is assumed to be constant along the whole arc, the chimney being flexible enough to justify this assumption. The direction of the wind assumed for calculations is of little importance, but a symmetrical direction makes working easier.

While this method may appear rather unsatisfactory, its results bear comparison with more elaborately obtained designs, and many bases constructed in accordance with these principles have proved successful. Moreover it should be remembered that, according to the intensity of the wind, there is an infinite number of positions of the C.P. to give equilibrium at any particular instant, and that calculations do no more than give the designer some assurance that equilibrium can be obtained when the worst conditions prevail. For an alternative method see Appendix 2.

Holding Down Bolts.

Turning now to the chimney in question, the self-weight is 15·3 + 7·4, or 22·7 tons, as previously calculated, plus, say, 0·8 tons for the base plate, giving a total of 23·5 tons. Similarly the overturning moment due to the wind is 8,490 inch tons, or 707·5 foot tons, for 100% strength. To suit B.S.S. 449, 150% wind must be taken, therefore the overturning moment for design purposes is, say, 1,060 ft. tons.

Assuming 16 holding down bolts each of core area A square inches and with a maximum stress of 5 tons per square inch, the moment of the 9 considered to be effective about the C.P. is, using distances scaled from the base shown on Fig. 2A and working in foot ton units, $5A\,[\,(1 \times 10{\cdot}2) + (2 \times 9{\cdot}75) + (2 \times 8{\cdot}6) + (2 \times 6{\cdot}80) + (2 \times 4{\cdot}75)\,] = 350A$ ft. tons approx.† Also the moment of the chimney weight about the C.P. is $23{\cdot}5 \times 4{\cdot}75$ or 112 ft. tons.

Therefore $350A + 112 = 1060$, or $A = 948 \div 350 = 2{\cdot}71$ sq. ins. A 2¼″ dia. bolt has a core area of 2·925 square inches, so that 16 bolts 2¼″ dia. will be suitable. The 7 other bolts not in tension easily take the horizontal shear.

The pull exerted by one bolt $= 2{\cdot}71 \times 5 = 13{\cdot}55$ tons or 30,400 pounds. Taking the bond stress between steel and concrete to be 100 pounds per square inch, each 1″ length of bolt carries a load of $\pi \times 2{\cdot}25 \times 100$ or 706 pounds. Thus the minimum bolt length in the concrete, neglecting the washer plates, is $30{,}400 \div 706 = 43''$, say 3′ 9″. 8″ square × ½″ thick washer plates will be fitted.

† The formula Bolt Leverage = $32 \times$ P.C.D. $\times$ A may be deduced for 16 H.D. Bolts.

Base Plate.

The pull exerted by all 9 bolts considered to be effective is 13·55 × 9 = 122 tons, thus the load carried by the leeward arc is 122 + 23·5 = 146 tons approx. Assuming the length of the arc exerting pressure on the concrete to be 8′ 0″ and the pressure per square foot to be 15 tons, the width of base plate required is $\frac{146}{8 \times 15}$ = 1·22 ft., or say, 15″. Also the load per foot run of arc is 146÷8, say, 18 tons.

If the base plate is divided into 8 equal portions, the joints (whose strength is neglected) coming midway between alternate holding down bolts, the length of each portion, measured along a 10′ 6″ diameter circle, is $\frac{10' \, 6'' \times \pi}{8}$ or 4′ 1½″. Also the centres of the holding down bolts are 2′ 0¾″ when measured along this circle. Thus if a pair of gussets at 6½″ centres is placed symmetrically at each holding down bolt, the following dimensions, measured along the 10′ 6″ diameter circle, result :—

(*a*) Joint in base plate to centre of nearest gusset = (4′ 1½″ ÷ 4) − (6½″ ÷ 2) = 9¼″ approx.

(*b*) Adjacent gusset centres (4′ 1½″ ÷ 2) − 6½″ = 18½″ approx.

Assuming the length (*a*) of the base plate to act as a cantilever and the length (*b*) as a freely supported beam—the gussets only acting as supports—the maximum bending moment in inch tons over each length resulting from a uniformly distributed load of W tons per square inch, is, working on a strip 1″ wide, $\frac{W \times 18\frac{1}{2}^2}{8}$.

Also assuming an unsupported length of 15″÷2 or 7½″ of the base plate (this dimension being measured radially) on either side of the 10′ 6″ diameter circle, the maximum bending moment on a strip 1″ wide resulting from the loading W above is $\frac{W \times 7\frac{1}{2}^2}{2}$ ins. tons, the 10′ 6″ diameter plate only acting as a support.

Now the ratio of these two maximum bending moments is

$$\frac{W \times 18\frac{1}{2}^2}{8} : \frac{W \times 7\frac{1}{2}^2}{2} \text{ or } 342{\cdot}25 : 225 \text{ or, say, } 1{\cdot}5 : 1.$$

Therefore to make the maximum bending stresses in the circumferential and radial fibres of the base plate equal, it is necessary to assume that the intensities of loading in the two directions are in the ratio 1 : 1·5. In other words, in this particular instance a loading of 6 tons per square foot should be taken to act over the

18½″ long plate, and a loading of 9 tons per square foot over the 7½″ length. In both cases the maximum bending moment on a 1″ wide strip is $\frac{9}{144} \times \frac{7\frac{1}{2}^2}{2}$ =, say, 1·8 ins. tons. If the thickness of the plate be t'' and the permitted stress 10 tons per square inch, since the modulus of section of a 1″ wide strip of plate $= \frac{t^2}{6}$, then $1{\cdot}8 = \frac{t^2 \times 10}{6}$ or $t^2 = 1{\cdot}08$, whence $t = 1{\cdot}04''$, for which a practical thickness of 1⅛″ would be ample.

The interaction of strains at right angles to each other and the stiffening effect of any base angles is neglected, partly for ease of working, and partly to allow for the effects of any drilling and for the fact that the use of an octagonal base plate involves some modification to the dimensions originally assumed.

Gussets.

With respect to the gussets, one pair at 6¾″ nominal centres will be fitted symmetrically on the outside of the chimney at each holding down bolt, and 16 such pairs will be fitted at corresponding points inside the chimney. Since the holding down bolts are pitched at 2′ 0¾″ centres and the load per foot run of arc is 18 tons, as previously calculated, each external or each internal pair will, at the worst, resist a thrust of $\frac{18 \times 2}{2} = 18$ tons at an average leverage of 7½″ ÷ 2 or 3¾″. Thus each single gusset should be suitable for a thrust of 9 tons at an average leverage of say 4″. The external and internal gussets together exert a central force on the chimney plate and no bending arises.

Now one ¾″ rivet in ⅜″ plate at 7·5 tons per square inch bearing stress can take a load of $7{\cdot}5 \times \frac{13}{16} \times \frac{3}{8} = 2{\cdot}3$ tons, and one ¾″ rivet at 4 tons per square inch tensile stress can take a pull of 0·518 × 4 or, say, 2 tons. Assuming ¾″ thick gussets each with 3″ × 3″ × ⅜″ angles each side for fixing to the chimney, 8 – ¾″ rivets passing through the ⅜″ chimney plate and through each angle will resist a shear of 8 × 2·3 = 18·4 tons, which is slightly more than the 2 × 9 = 18 tons imposed by one external and one internal gusset acting together. Also 2 rivets in tension at the top of the gusset and 2 at the bottom with centres of 18″ will resist a bending moment of 2 × 2 × 18 = 18 × 4 ins. tons per gusset, which is twice that imposed by 9 tons at 4″ average leverage acting on each gusset. This means that the leg of the 3″ × 3″ × ⅜″ angle which is attached to the chimney must be long enough for 6 rivets. Thus if the gussets and angles extend for a height of 2′ 0″, ample space will be available. The rivets fixing the angles to the gussets will be 6 at least in number, all acting in double shear and bearing on $\frac{3}{4}''$ thickness of material, and they are ample for the load of 9 tons per gusset.

As a check on the gussets, it will be assumed that a load of 5 tons acts in compression on the unstiffened outer portions extending from the edges of the base plate to the top of the gussets—the length of these portions being, say, 25″. Now the radius of gyration of $\frac{3}{4}''$ thick plate, calculated about the centre of the thickness, is $0{\cdot}75 \div \sqrt{12} = 0{\cdot}217$, therefore, $l/r = 25 \div 0{\cdot}217 = 115$. Taking the permitted stress of 3·5 tons for this ratio of slenderness, the width of plate required is $\frac{5}{3{\cdot}5 \times 0{\cdot}75}$ = say, 2″, which is reasonable.

The internal gussets can be cut to the usual triangular shape, but the external ones should project far enough at the top to carry the seating pads of the holding down bolts, the unsupported edges of the gussets being tied by a channel cleat or equal. Since the holding down bolts each exert a force of 13·55 tons at, say, 4″ leverage on each pair of gussets, as against the 18 tons already assumed, no check on strength is necessary.*

With regard to the seating pads, these carry a load of 13·55 tons at the centre of a $6\frac{3}{4}''$ span. Thus the maximum bending moment is $13{\cdot}55 \times \frac{6{\cdot}75}{4}$ or 23 ins. tons, ignoring the spread of the bolt load. Taking $2\frac{1}{2}''$ thick plate and assuming a stress of 10 tons per square inch, the nett width of plate required is $\frac{23}{10} \div \frac{2{\cdot}5^2}{6}$ $= 2{\cdot}2''$. Thus a pad 6″ wide will make ample allowance for the $2\frac{3}{8}''$ dia. hole required for the holding down bolt. It is probably advisable for the seating pads to be left loose to allow for any inaccuracies in drilling, etc., and the pads should be about 9″ long. Fig 2A shows a suitable arrangement of gussets.

Angle Ring at Base.

To connect the bottom strake of the chimney to the base plate it is necessary to fit one (or in larger chimneys two) angle ring splayed to suit the taper of the cone. The horizontal leg bears directly on the base plate to transmit compression, and the gussets mentioned earlier take the pull from the holding down bolts. Thus the rivets connecting this leg to the base plate do not require calculating, and can be at a nominal pitch of say 3″. The rivets connecting the vertical leg to the chimney strake must be suitable for transmitting the compression arc loading of 18 tons per foot run as previously calculated, and, working on a load carrying capacity of 2·3 tons per $\frac{3}{4}''$ rivet, some 8 rivets per foot run of angle are required. Double riveting at 3″ pitch will suffice, and the pitching determined earlier for 140′ 0″ down must be modified to suit this local condition. It can be assumed that the concentrated loading at base level is quickly dispersed and does not affect the next higher seam.

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* See also Appendix 2, Part *(c)*.

The local plate stress due to the loading of 18 tons per foot run is $18 \div 12 \times \frac{3}{8}'' = 4$ tons per square inch. This can be accepted, since it results from 150% wind and the plate is stiffened by the gussets and angle ring.

A $7'' \times 4'' \times \frac{3}{4}''$ angle, the 4″ leg horizontal, will be suitable for the base ring and allows the following dimensions :—underside of angle to lower row of rivets, $2\frac{3}{4}''$; distance between rows of rivets, 3″ ; upper row of rivets to toe of angle, $1\frac{1}{4}''$. All components of the chimney base should be carefully bedded, packs being used wherever necessary. Simple butt straps can be used to connect the octagonal sections of the base plate together.

The chimney base designed above is for a completely riveted stack, but it is common practice to use an all welded base fabricated in the shop and despatched in convenient sections. The same principles of design can be employed, but it is advisable to use a thicker bottom strake—say $\frac{1}{2}''$ in this instance—and make it about 2′ 6″ high. The depth of the gussets can be reduced, and the base and gusset angles dispensed with—all effecting economies.

Concrete Block.

With respect to the concrete base block, a circular shape will be assumed, although any regular polygon is possible to achieve saving in shuttering. The density of concrete will be taken as 140 lbs. per cubic foot, and the maximum loading on the earth as 2 tons per square foot. Furthermore, the compressive stress W/A should exceed the tensile stress M/Z on the windward side for designs to more conservative principles.

To arrive at the minimum diameter of the base, the value of W/A must be approximately 1 ton per square foot, which would result from a depth of concrete of 2240 ÷ 140 or 16′ 0″, and the value of M/Z must be slightly under 1 ton per square foot. Now the projected area of the chimney exposed to the wind is $7 \times 140 + 3{\cdot}5 \times 15 = 980 + 52{\cdot}5 = 1032{\cdot}5$ square feet, and the total 100% wind is $1032{\cdot}5 \times \dfrac{23}{2240}$ or 10·6 tons. As calculated previously, the 100% overturning moment is 707·5 foot tons, so that the point of action of the wind above the base level is 707·5 ÷ 10·6 or 67′ 0″.

With a base block 16′ 0″ deep the leverage of the wind is 67 + 16 or 83 feet, so that the overturning moment for 100% wind is 10·6 × 83 or 880 foot tons at the underside of the base, and that for 150% wind 1320 foot tons.

With Z as $\frac{\pi r^3}{4}$ ft. units, 1,320 ÷ $\frac{\pi r^3}{4}$ is roughly equal to 1 ton per square foot, or $r^3 = 1{,}680$, whence $r = 11{\cdot}88$, r being the radius of the base block in feet. Thus a solid concrete block 16′ 0″ deep × 24′ 0″ diameter is satisfactory, and has a weight of 16 × 144 × π × 140 ÷ 2,240 or 450 tons, which is some twenty times the weight of the steel chimney, whose effect on the W/A stress can be neglected.

If the diameter of the base block is not limited, it might be better and cheaper to assume a depth of say 6′ 0″. Then W/A is 140 × 6 ÷ 2,240 or 0·375 tons per square foot. Also the overturning moment at the underside of the base block for 150% wind is 10·6 × 1·5 × (67 + 6) or 1,160 foot tons, so that $\frac{\pi r^3}{4} \times 0{\cdot}375 =$ 1,160, whence $r^3 = 3{,}940$. This gives a value of 32′ 0″ diameter and a weight of 6 × 256 π × 140 ÷ 2240 = 300 tons.

The block shown in Fig. 2 is yet another size as calculated by the above conservative design. See also part (*a*) of Appendix 3.

Alternatively, when the client will consent or when a more competitive block is required, the "no tension" method may be used to arrive at a smaller size of block.

Referring to this method as explained in part (*b*) of Appendix 3, and assuming again a value of 6′ 0″ for H, the approximate method for a circular block leads to the equation :—

$$6 \times D^3 \div 69{\cdot}2 = 1160 \text{ (as already calculated).}$$

From this $D^3 = 13{,}400$ or D = 23·75, say 24′ 0″.

The weight of such a block is $6 \times 24^2 \div 20{\cdot}4 = 170$ tons, and the chimney weighs 23·5 tons (as already calculated). Thus the total weight is 193·5 tons and the approximate maximum stress is 193·5 ÷ K or $193{\cdot}5 \div \frac{24^2}{6} = 2{\cdot}02$ tons per square foot, which can be accepted since it is known to be higher than the actual stress.

To check this stress by the $\frac{W}{A} + \frac{M}{Z}$ method, $A = \frac{24^2}{2{\cdot}55} = 226$, $Z = \frac{24^3}{42} = 330$, and $e = \frac{24}{4{\cdot}72} = 5{\cdot}1$. Thus $\frac{W}{A} = \frac{193{\cdot}5}{226}$ = 0·86 tons per square foot. Also the moment of 1160 foot tons must be reduced by 193·5 × e or 985, leaving a nett value of 175 foot tons for M. Therefore $\frac{M}{Z} = \frac{175}{330} = 0{\cdot}53$ compression (0·39 tension). The resultant stresses are :—leeward, 1·39 tons per square foot ; windward, 0·47. Thus the base is satisfactory.

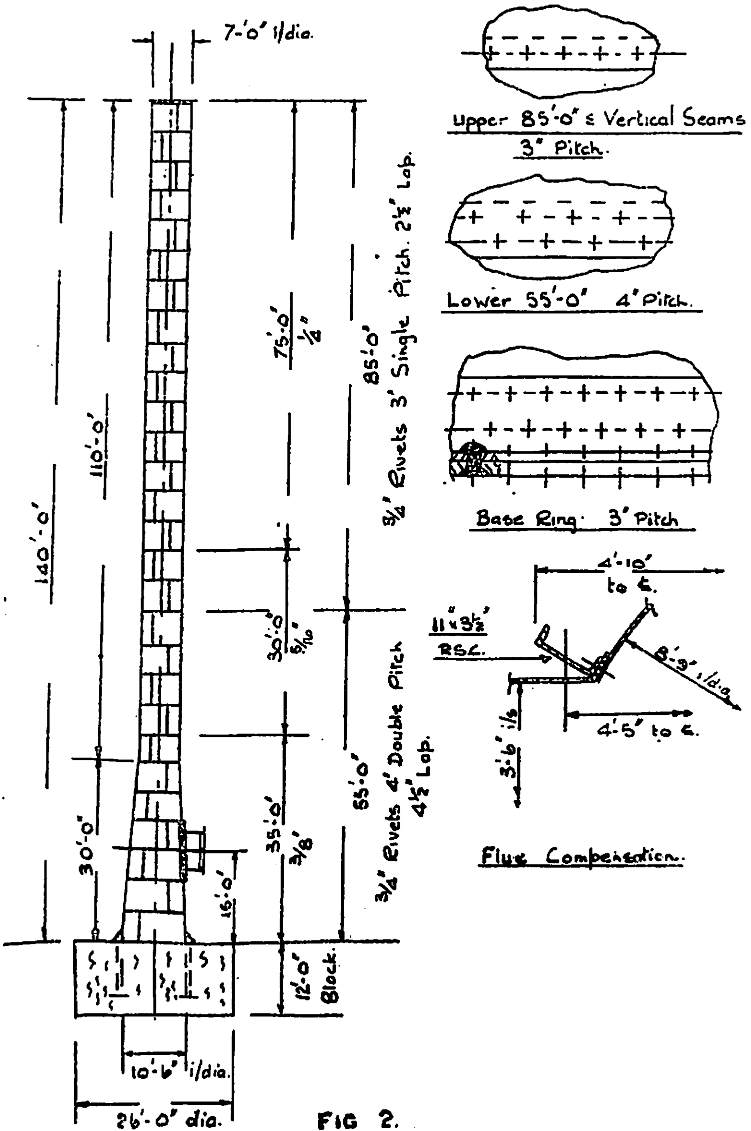

FIG 2.

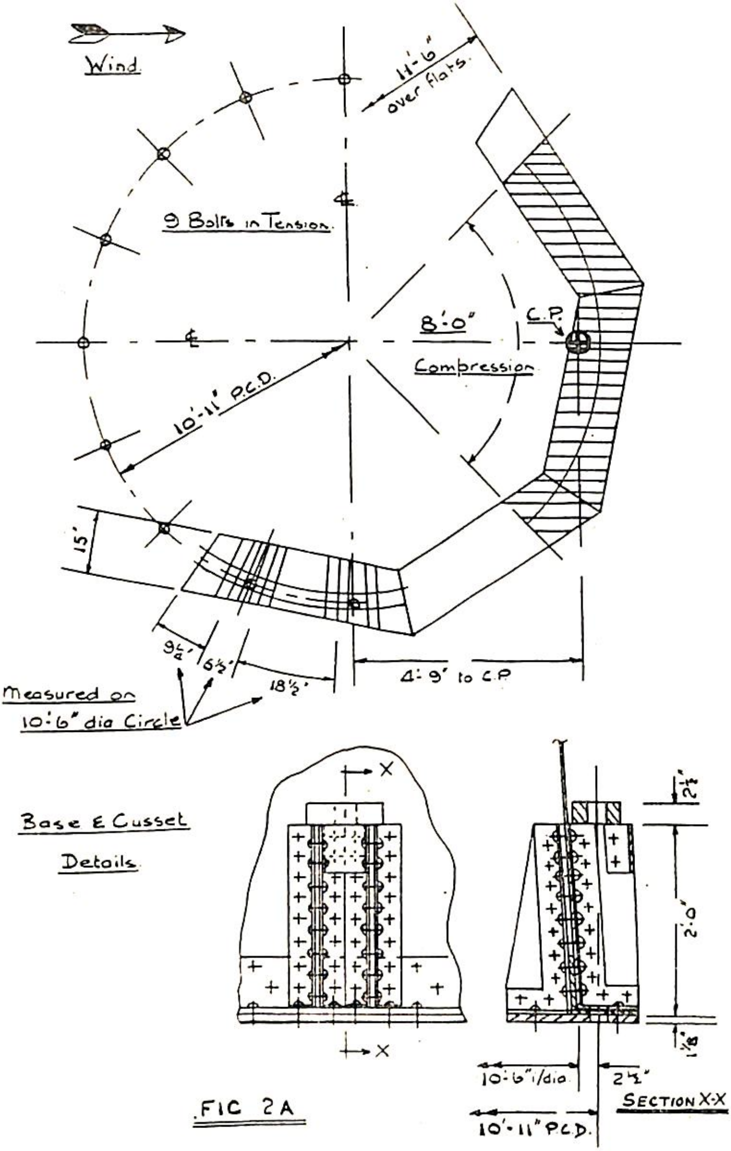

FIG 2A

If, when the approximate method was used, the value of $W_c \times E$ had been increased by, say, 20% to allow for the weight of the chimney, the value of D^3 would have been $13{,}400 \times \dfrac{100}{120} = 11{,}200$

from which a value for D of 22′ 6″ would have been derived. The weight of such a base is $6 \times 22{\cdot}5^2 \div 20{\cdot}4 = 150$ tons, so that the total weight would have been 173·5 tons, and the maximum stress (by the approximate method) $173{\cdot}5 \times 6 \div 22{\cdot}5^2 = 2{\cdot}05$ tons per square foot.

The corresponding maximum and minimum stresses as checked by the $\frac{W}{A} + \frac{M}{Z}$ method for the "no tension" design would have been 2·1 and 0 tons per square foot respectively.

The block 6′ 0″ deep × 24′ 0″ diameter and weighing 170 tons would be suitable, but a size 6′ 0″ deep × 25′ 0″ diameter and weighing 185 tons is suggested in this instance because of the extra safety provided.

In this first example of chimney design, an attempt has been made to cover the main features of all steel chimneys, and in succeeding examples only additional points will be discussed at any length.

(2) Self-Supporting Lined Chimney 12′ 0″ I/Dia. × 200′ 0″ High.

Specification.—Riveted design; $\frac{5}{16}$″ minimum thickness; preferably parallel over full height; lined throughout with $4\frac{1}{2}$″ firebrick; supported on steel beams forming part of boiler house structure. Suitable for temporary condition of 20 lbs. per square foot wind on the full 200′ 0″ of projected area when unlined, and for permanent condition of 30 lbs. wind on the upper 140′ 0″ of projected area when lined. Maximum rivet stresses:—shear, 5 tons per square inch; bearing, 10 tons per square inch. Compressive plate stress limited to 3 tons per square inch except locally. (See Figs. 3 and 3A).

In this example the plate thicknesses will be read off from a graph which combines the $\frac{M}{Z}$ and $\frac{W}{A}$ diagrams for the two specified conditions—the notation being as for Example (1). For the purposes of the graph it will be assumed for self-weight that the chimney is $\frac{5}{8}$″ thick throughout, and for the values Z and A that the chimney is $\frac{1}{2}$″ thick throughout. Any error due to the first assumption will be slight, although some experience is required to make a suitable choice. For the $\frac{M}{Z}$ and $\frac{W}{A}$ diagrams, which will be drawn to scale for the assumed $\frac{1}{2}$″ thickness, it is necessary to measure the graph to appropriate scales to find the actual stresses in plates of other real thicknesses. Thus if a scale of 1″ to 1 ton per square inch were used for $\frac{1}{2}$″ plate, then a scale of $\frac{3}{4}$″

to 1 ton per square inch would be required for $\frac{3}{8}''$ plate. Alternatively the stress scaled off from the graph as drawn could in this instance be multiplied by the ratio $\frac{1}{2}''$ to $\frac{3}{8}''$ to give the actual stress in $\frac{3}{8}''$ plate. The above arbitrary constant thicknesses are chosen so that the W/A diagram is a straight line, and so that the M/Z diagram over the parallel portion of the chimney takes the same form as the diagram of M, the bending moment due to wind, because Z is constant.

The stress resulting from a 12″ wide strip of $\frac{5}{8}''$ thick plate 200′ 0″ high and resting on a 12″ × $\frac{1}{2}''$ section is, when 30% is added to the weight of the plate to cover for laps and fittings $\dfrac{25{\cdot}5 \times 1{\cdot}3 \times 200}{12 \times \frac{1}{2} \times 2240} = 0{\cdot}50$ tons per square inch approx. Thus the W/A stress at the base due to the weight of steel is determined, and, since the load increases uniformly for constant plate thickness, the W/A diagram is a straight line—shown as 0 - 1 on Fig. 3.

The annular cross section of a firebrick lining 12′ 0″ o/dia. × 11′ 3″ i/dia. (as read from tables) is 113·1 − 99·4 = 13·7 square feet, so that, with a density of 140 lbs. per cubic foot or 16 cubic feet to the ton, the total weight of the lining is 13·7 × 200 ÷ 16 = 171 tons. The cross sectional area of the steel is $144 \times \pi \div 2 = 226$ square inches, so that the W/A stress for brickwork is 171 ÷ 226 = 0·76 tons per square inch. The diagram—again a straight line—is represented by 0 − 2 on Fig. 3.

The wind load at 20 lbs. per square foot on an area 12′ × 200′ 0″ is 12 × 200 × 20 ÷ 2240 = 21·4 tons at a leverage of 100′ 0″, giving a bending moment of 25,680 inch tons at the base, the diagram being parabolic. Now the Z value is $\pi \times 72^2 \times \frac{1}{2} = 8{,}143$ inch units, so that the maximum M/Z value is 25,680 ÷ 8,143 = 3·16 tons per square inch. Likewise the wind load at 30 lbs. per square foot on the upper 140′ 0″ is 12 × 140 × 30 ÷ 2,240 = 22·5 tons, giving at 140′ 0″ down a bending moment of 22·5 × 840 or 18,900 inch tons, and at 200′ 0″ down a value of 22·5 × 130 × 12 = 35,000 inch tons. Thus the M/Z value at 140′ 0″ down is 18,900 ÷ 8,143 = 2·32 tons per square inch—the diagram being parabolic so far—and 35,000 ÷ 8,143 = 4·3 tons per square inch—the diagram increasing uniformly from 140′ 0″ down—at the base. The diagrams for 20 and 30 lbs. wind are represented by curves 0 - 3 and 0 - 4 respectively on Fig. 3.

A given point C (on the 20 lbs. diagram) occurs at the intersection of line OB and the horizontal line through A, where OA = KOD and DB = KD 3, K being here four-fifths of OD, the full height, and of D3, the M/Z value at the base. The 30 lbs. diagram is plotted in a similar manner as far as 140′ 0″ down, where it becomes a straight line.

Turning now to reading the stress graph, 3·0 tons in actual $\frac{5}{16}''$ plate measures $3{\cdot}0 \times \frac{5}{8}$ or 1·875 tons on the graph for the theoretical $\frac{1}{2}''$ plate. Likewise 3·0 tons in actual thicknesses of $\frac{3}{8}''$, $\frac{1}{2}''$, $\frac{5}{8}''$, $\frac{3}{4}''$, $\frac{7}{8}''$ and 1", measures on the graph as drawn 2·25, 3·00, 3·75, 4·5, 5·25 and 6·0 tons respectively. Starting with the minimum thickness of $\frac{5}{16}''$, it is found that the graph scales 1·875 tons 100′ 0″ down, so that $\frac{5}{16}''$ plate is suitable over the first 100′ 0″. The ranges for other thicknesses are determined in the same way, and thicknesses for the whole chimney are shown on Figure 3. Distances are taken to the nearest 5′ 0″ above the point of maximum permitted stress, and only the result of 30 lbs. wind plus lined weight is considered, the temporary condition of 20 lbs. wind without lining clearly giving lower stresses.

The approximate weight of steel in the chimney as designed so far with $\frac{1}{16}''$ plate at $1{\cdot}3 \times 2{\cdot}55$ lbs. per square foot is $1{\cdot}3 \times 2{\cdot}55 (5 \times 100 + 6 \times 10 + 8 \times 20 + 10 \times 20 + 12 \times 20 + 14 \times 20 + 16 \times 10) \times 12 \times \pi$lbs. $= 125 \times 1600 \div 2240 = 89{\cdot}5$ tons, which gives a W/A stress for steel at the base of $89{\cdot}5 \div 226 = 0{\cdot}40$ tons per square inch against the 0·50 assumed. This discrepancy can be ignored, since the effect of the steel weight on the total stress is obviously small. The weight of the base will be taken as 1·5 tons.

Holding Down Bolts.

In calculating the holding down bolts, 16 pitches will be assumed—nine only being effective—on a 12′ 0″ dia. circle, each pitch having a core area of A square inches ; and the compression will be assumed to act at five points where the chimney rests on the supporting steelwork (shown in chain dot on Fig. 3A) with a centre of pressure 5′ 0″ from the centre of the chimney. Proceeding as in Example 1 with a bolt stress of 5 tons per square inch, for the unlined condition, where the weight of steel is 91·0 tons, *i.e.*, 89·5 + 1·5, and the overturning moment 21·4 × 100 or 2,140 ft. tons,

$$5A\,(11 + 2 \times 10{\cdot}5 + 2 \times 9{\cdot}25 + 2 \times 7{\cdot}4 + 2 \times 5) + 91 \times 5 = 2140$$

$$\text{or } 376{\cdot}5A = 1685, \text{ whence } A = 4{\cdot}48 \text{ square inches.}$$

The total compression load is $91{\cdot}0 + 9 \times 5 \times 4{\cdot}48 = 293{\cdot}0$ tons.

Likewise for the lined condition, where the total weight is 91·0 + 171 or 262·0 tons—it being assumed that all the brickwork is carried by the steel chimney—and the bending moment is 22·5 × 130, or 2930 ft. tons.

$$376{\cdot}5A + 262{\cdot}0 \times 5 = 2930 \text{ or } 376{\cdot}5A = 1620{\cdot}0.$$

Therefore $A = 4{\cdot}30$ square inches, and the total compression load is $262 + 9 \times 5 \times 4{\cdot}30$ or 456 tons.

For the bolts, 2 – 2″ diameter bolts having a total core area of $2 \times 2{\cdot}31$ or 4·62 square inches will suffice at each pitch, one inside and one outside the chimney and, if possible, pitched to clear the flanges of the steel supporting beams, fastening being effected by clamping members passing under these beams. The compression load of 456 tons imposes a load of 91·2 tons per point.

If each point is only 6" × 6", the stress is 91·2 ÷ 36 = 2·54 tons per square inch, which can readily be taken by the steel.

Base.

A suggested base comprises an octagonal base plate 15" wide × 1" thick with 4 gussets 7" wide × 1" thick × 20" high per pair of bolts, the gussets being at 4" centres and covered by 2" thick seating pads for the 2 – 2" diameter holding down bolts at, say, 9" cross centres. A detail assuming welded design is shown on Fig. 3A. If the load of 91·2 tons per point is taken by the 4 gussets and, say, 4" of chimney plate, then the compressive stress is 91·2 ÷ 32 = 2·85 tons per square inch, which is well within the capacity of the steel. The holding down bolt pull per pitch is 4·48 × 5 = 22·4 tons, which is readily taken by the above gussets and transferred to the chimney plate, no bending resulting because the load is balanced. Also if an angle of dispersal of 30° to the vertical in the chimney plate of the gusset loads is assumed, the load of 91·2 tons is taken by a length of 1" plate some 27" long, *i.e.*, the plate stress is 91·2 ÷ 27 = 3·38 tons per square inch. This stress, being local, is probably acceptable, although it might be better to increase the bottom strake to $1\frac{1}{8}$" thick and reduce its height to, say, 2' 6" instead of the standard 5' 0" assumed.

Bricking rings in, say, 4" × 3" × $\frac{3}{8}$" angle would be required inside the chimney at about 10' 0" centres for the full height, so positioned as to miss the strake joints. A typical section is shown on Fig. 3.

With respect to rivets it will be assumed that $\frac{3}{4}$" rivets can be used for thicknesses up to $\frac{1}{2}$", and $\frac{7}{8}$" rivets for thicker plates. For the $\frac{5}{16}$" plate, Fig. 1 shows that bearing is the criterion, and suitable pitches can be read from Table 3. For the thicker plates shear is the criterion and Tables 1 and 2 should be consulted. For instance from Table 2, for $\frac{7}{8}$" plate stressed to 3·0 tons per square inch, $\frac{7}{8}$" rivets at $1\frac{1}{4}$" equivalent pitch are suitable, *i.e.*, treble rivets at $3\frac{3}{4}$" pitch and $2 \times (1\frac{1}{2} + 2\frac{1}{4})$ or $7\frac{1}{2}$" lap. Checking this, the load per rivet = $3{\cdot}0 \times \frac{7}{8} \times 1{\cdot}25$ = 3·28 tons on a shear area of 0·69 square inches, *i.e.*, shear stress = 3·28 ÷ 0·69 = 4·75 tons per square inch. The riveting for vertical seams can be taken as one row of rivets at 3" pitch.

Before leaving this example, the effects of coning out the chimney will be discussed. If space permitted, it would be more economical to cone out the lower 40' 0" to some 18' 0" diameter. This coning out also simplifies the design of the base because the loading intensity is reduced.

To draw the graph stresses in the cone, it is necessary to calculate, as in Example 1, the stresses at several points down the cone and then plot these. In general the W/A stress diagram tapers inwards over the cone length and the M/Z stress diagram turns sharply at the beginning of the cone and then follows a curve towards the base stress.

In this particular instance for the 30 lbs. wind condition, the value of Z at 18′ 0″ diameter and $\frac{1}{2}''$ plate is $\pi \times 108^2 \times \frac{1}{2} = 18{,}300$ inch units, so that the M/Z stress at the base is $35{,}000 \div 18{,}300 = 1{\cdot}92$ tons per square inch. Similarly if the weight of steel

Compressive Stress

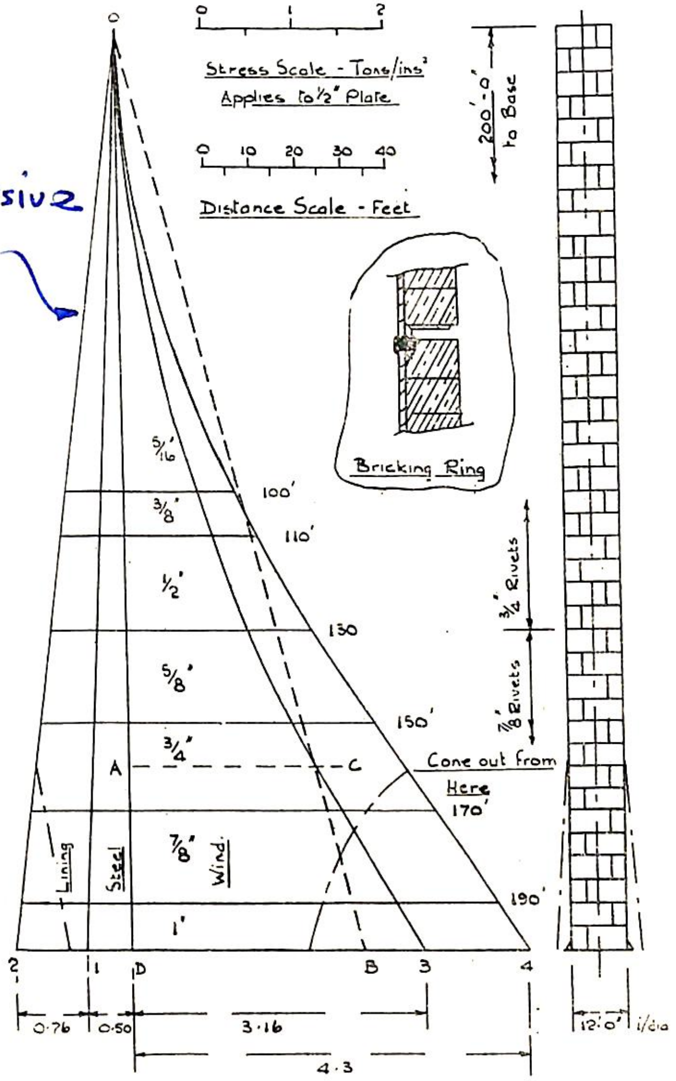

FIG 3

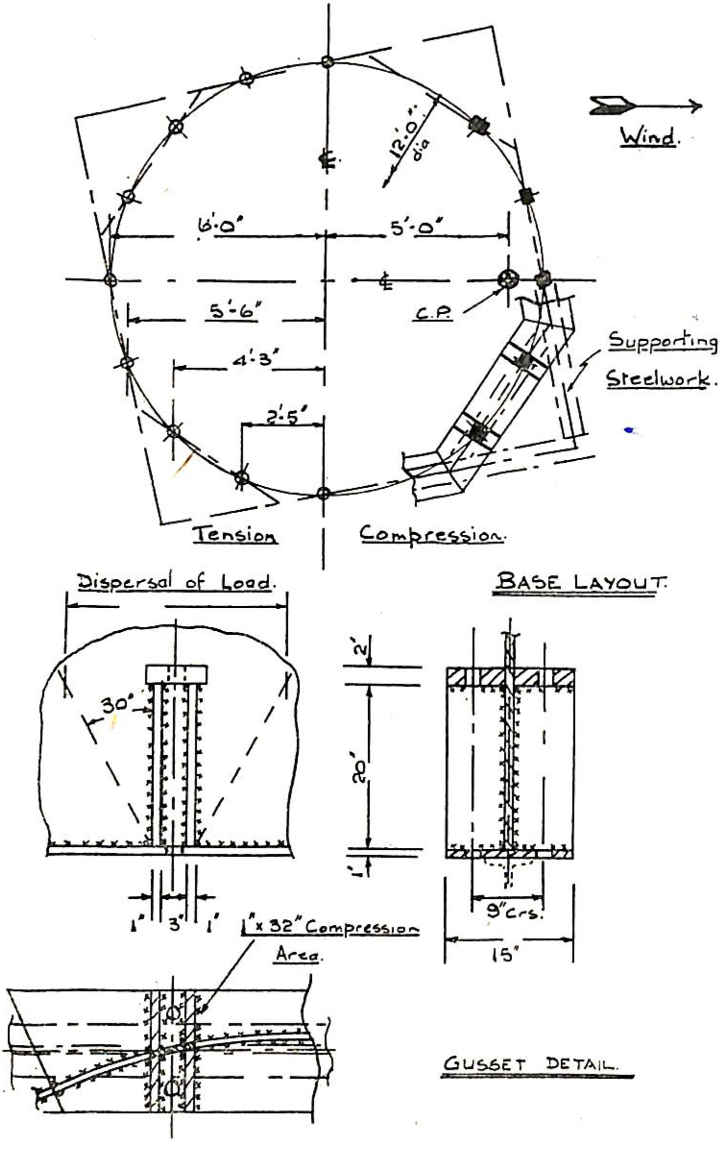
Wind.
12'-0" dia
℄
6'-0"
5'-0"
5'-6"
℄
C.P.
4'-3"
2'-5"
Supporting Steelwork.
Tension
Compression.
Dispersal of Load.
BASE LAYOUT.
30°
2"
20"
1"
1" 3" 1"
1" x 32" Compression Area.
9" crs.
15"
GUSSET DETAIL.

FIG 3A.

is taken as 95 tons to allow for the cone, and the weight of brickwork calculated only over the upper 160′ 0″ (the coned out portion being self-supporting), the total weight is $95 + (171 \times 160/200)$, or 232 tons, acting on a cross sectional area of $216 \times \pi \div 2$ or 339 square inches. Thus the total W/A value is $232 \div 339 = 0{\cdot}685$ tons per square inch.

These values are plotted in chain dot on Fig. 3, and intermediate values are assumed in this instance, although in actual practice they should be computed. Examination shows that the thickness of ¾″ might be maintained for all the cone.

It will be clear that the graphical method of determining plate thicknesses emphasises the relative unimportance of W/A, especially when a chimney is unlined, and also indicates the value of the coned out lower portion, showing in fact at what level it may be necessary to commence the cone to avoid excessively thick plates.

(3) Guyed Unlined Chimney 5′ 0″ I/Dia. × 100′ 0″ High.

Specification.—Located in S.E. England; conditions of maximum exposure; full height exposed to wind, which should be to B.S.S. 449, 1948. Parallel over full height; supported by 3 equally spaced guys at 45° to vertical at 70′ 0″ level; riveted design. Maximum stresses in rivets:—shear, 5 tons per square inch; bearing, 10 tons per square inch. (See Fig. 4).

The wind force per square foot of projected area to suit the above conditions, as taken from B.S.S. 449, is $24{\cdot}8 \times 0{\cdot}7 = 17{\cdot}36$ lbs, say 18 lbs, at 100% intensity. Thus the total wind load is $5 \times 100 \times 18 \div 2240 = 4$ tons, and the bending moment at the base, neglecting the effect of the guy ropes, is $4 \times 600 = 2400$ inch tons.

If the chimney is assumed to be freely hinged at the base, the resultant horizontal guy force, P, is $4 \times 50 \div 70$ or 2·86 tons for 100% wind intensity, and there is no resultant bending moment at the base.

If the chimney is assumed to be perfectly fixed at the base and the guys prevent any deflection at the 70′ 0″ level, then, taking the moment of inertia of the chimney to be constant, the first moment of area of the bending moment diagram resulting from the wind equals the first moment of area of the bending moment diagram resulting from force P, both moments of area being taken about the 70′ 0″ level and over the range 70′ 0″ to 0′ 0″, *i.e.*, base to guy level. (See text-books on strength of

over

ERRATUM

"NOTES ON THE DESIGN OF MILD STEEL CHIMNEYS"

A.E.S.D. members who purchased a copy of the technical pamphlet entitled "Notes on the Design of Mild Steel Chimneys," are asked to note the following correction:—

The equation at the foot of page 37 should read:—

$$\frac{0{\cdot}04}{2}\int_0^{70}(30+x)^2\,x\,dx = \mathrm{P}\int_0^{70} x^2\,dx$$

and not as printed in the text.

$$\text{or } 0{\cdot}02\left(\frac{900x^2}{2}+\frac{60x^3}{3}+\frac{x^4}{4}\right)_0^{70}=\text{P}\left(\frac{x^3}{3}\right)_0^{70}$$

$$\text{or } 0{\cdot}02\left(\frac{450}{x}+20+\frac{x}{4}\right)_0^{70}=\frac{\text{P}}{3}$$

whence $\text{P}=0{\cdot}06\,(6{\cdot}43+20+17{\cdot}5)=2{\cdot}63$ tons for 100% wind.

The restraining bending moment due to P is $2{\cdot}63\times 840$, or 2210 inch tons, leaving a nett value at the base of 190 inch tons.

In practice a guyed chimney is neither freely hinged nor perfectly fixed at the base, so that some compromise is necessary. For the present example P will be taken as 2·8 tons when calculating the forces in the guy ropes, and a resultant bending moment of 100 inch tons will be assumed at the base, both values being for 100% wind.

If the chimney is $\frac{1}{4}''$ thick throughout, the value of Z is $\pi\times 30^2\div 4=707$ inch units, so that the M/Z values when $\text{M}=2400$ and 2300 inch tons respectively are 3·4 and 3·26 tons per square inch. Likewise $\frac{\text{W}}{\text{A}}$ at the base is $\frac{10{\cdot}2\times 1{\cdot}3\times 100}{12\times\frac{1}{4}\times 2240}=0{\cdot}198$ tons per square inch.

With respect to the guy ropes, it is shown in Appendix 2, part (*d*), that, where the resultant horizontal guy force is P tons, the maximum horizontal component in any one of the three equally spaced ropes is 1·15 P, and that the maximum sum of the horizontal components in any two of the ropes is 2 P. From this it follows that, for an angle of 45° to the vertical for the guys, the maximum vertical component in any one guy is also 1·15 P, and the maximum sum of the vertical components in any two guys is also 2 P.

In calculating the compressive stresses resulting from the above vertical components, two cases (*a*) and (*b*), may be considered. In case (*a*) the guy ropes can be assumed to be fixed to a flat or angle band at the 70′ 0″ level, and the vertical component from each to be dispersed over one third of the circumference of the chimney. In case (*b*) the guy rope band is assumed to be some 2·5′ × tan 45° below the 70′ 0″ level, and the force in each rope to be resolved at the vertical axis of the chimney into a horizontal component at the 70′ 0″ level and a vertical component which exerts a uniformly distributed compressive force on the whole circumference.

The maximum stresses resulting from 100% wind intensity are for cases (*a*) and (*b*) respectively $(1{\cdot}15\times 2{\cdot}8)\div(20\times\pi\times\frac{1}{4})$ or 0·205 tons per square inch exerted on one-third of the windward side of the chimney, and $(2\times 2{\cdot}8)\div(60\times\pi\times\frac{1}{4})$ or 0·12 tons per square inch exerted over the whole circumference.

All the above stresses are shown graphically on Fig. 4, and the maximum stress is approximately 0·6 tons per square inch for 100% wind and ¼″ thick plate, so that the chimney is quite satisfactory in this thickness.

Base.

For the base the overturning moment at 150% wind intensity is assumed to be $100 \times 1{\cdot}5 \div 12 = 12{\cdot}5$ foot tons, and the weight of steel is approximately $\dfrac{10{\cdot}2 \times 1{\cdot}3 \times 100 \times 5\,\pi}{2240} = 9{\cdot}3$ tons, so that the chimney weight alone gives stability with a centre of pressure $\dfrac{12{\cdot}5}{9{\cdot}3}$ or 1·35 feet from the centre of the chimney. This is reasonable, so that theoretically no foundation bolts are required. In practice 8 bolts ⅞″ dia. are suggested, with a concrete base 5′ 0″ deep × say 8′ 0″ dia.

The total vertical load on the compression arc with 150% wind is $9{\cdot}3 + (2 \times 2{\cdot}8 \times 1{\cdot}5) = 17{\cdot}7$ tons on an area of $60 \times \pi \times \frac{1}{4} \div 4 = 11{\cdot}8$ square inches (for a 90° arc), giving a stress of 1·5 tons per square inch, or 4·5 tons per foot run. A base plate 6″ wide × 90° arc × ½″ thick gives an area of 1·96 square feet in contact with the concrete block, or a stress of $\dfrac{17{\cdot}7}{1{\cdot}96} = 9$ tons per square foot, which is suitable. Also the Z per foot run is $\dfrac{12 \times 1}{6 \times 4}$ or ½ inch units for a bending moment per foot run of $\dfrac{4{\cdot}5 \times 6}{8}$ or 3·4 inch tons. This gives a reasonable bending stress of 6·8 tons per square inch. The base plate should be reinforced with gussets adjacent to the bolts.

Guy Ropes and Anchor Blocks.

The maximum horizontal and vertical component in any one guy rope for 150% wind is $2{\cdot}8 \times 1{\cdot}15 \times 1{\cdot}5 = 4{\cdot}83$ tons. This means that the concrete anchor block, if friction alone is to resist the horizontal component, should weigh at least $4{\cdot}83\,(1 + 1/\mu)$ tons, where μ is the coefficient of friction between concrete and earth. When μ is taken as 0·25, the block should have a minimum weight of $4{\cdot}83 \times 5$ or say 25 tons—the size 10′ 0″ × 8′ 0″ × 5′ 0″ deep satisfying this requirement.

The resultant pull in the guy rope for this condition is $4{\cdot}83 \times \sqrt{2} = 6{\cdot}83$ tons, for which a wire rope with an ultimate breaking strength of 6 × 6·83 or 41 tons would be suitable. The fittings generally supplied include shackles, tension screw, thimbles, and clips, and all should be designed for the above load. The anchor bolt should be embedded deeply enough in the concrete block to resist the pull of 6·83 tons by means of adhesion between steel and concrete as calculated in Example 1, and its nett sectional area should be at least 6·83 ÷ 5 or 1·37 square inches.

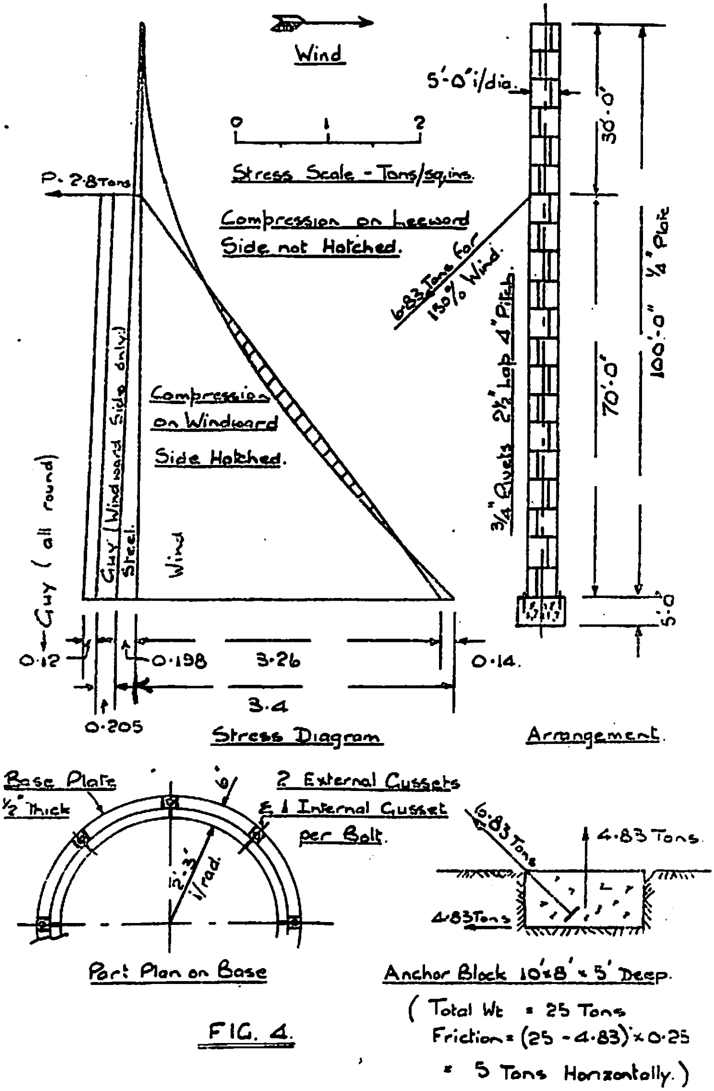

FIG. 4.

It will be seen that the use of guys to support a chimney effects economy in steel and concrete base block ; but, on the other hand, guy ropes cause obstructions and together with their anchorages take up much ground space.

(4) Self-Supporting Unlined Chimney 2′ 6″ I/Dia. × 50′ 0″ High.

The usual specification for such a chimney calls for 4 or 5 flanged sections suitable for bolting together at site ; for $\frac{3}{16}$″ or $\frac{1}{4}$″ thick plate ; and for a $\frac{1}{8}$″ thick conical cowl at the top. The small size of the section allows shop assembly, and welded design with flat bar or angle flanges is common. In general only the flange connections and the base require any calculations. (See Fig.5).

In this instance 30 lbs. per square foot wind will be assumed, and the thickness will be taken as $\frac{1}{4}$″ throughout.

At the base, assuming that the chimney is not coned out, the $\frac{W}{A}$ stress will be approximately $\frac{50 \times 10{\cdot}2 \times 1{\cdot}1}{12 \times \frac{1}{4} \times 2240}$ or 0·084 tons per square inch (adding 10% for fittings and flanges) and the total load is some $\frac{50 \times 10{\cdot}2 \times 1{\cdot}1 \times 2{\cdot}5\ \pi}{2240}$ or 2 tons.

The wind load on 50′ 0″ is $\frac{2{\cdot}5 \times 50 \times 30}{2240} = 1{\cdot}67$ tons, giving a bending moment at the base of $1{\cdot}67 \times 25 \times 12$, or 500 inch tons. The modulus of section being $\pi \times 15^2 \times \frac{1}{4}$ or 177 ins. units, the M/Z stress is 2·82 tons per square inch, and the total maximum stress is $2{\cdot}82 + 0{\cdot}084$, or roughly 3 tons per square inch, which is acceptable.

Flanged Joint.

If the base has an angle flange (to connect to the flange of a boiler-hood for instance), it will be assumed that the centre of pressure between the flanges is at 12″ from the axis of the chimney, that the bolts in tension are equivalent to a semi-circle of thickness t″ and radius say 16″, and that the stress on such a semi-circle is 5 tons per square inch at all points. These assumptions are similar to those made in Example (1).

The centre of gravity of a semi-circle as assumed is $\frac{2 \times 16}{\pi}$ or 10·2″ from the axis of the chimney, so that the distance from centre of gravity to centre of pressure is $10{\cdot}2 + 12$ or 22·2″. The area of such a semi-circle is $16 \times \pi \times t$ ins.^{2}, so that the tensile load is $5 \times 16 \times \pi \times t$ or $250t$ tons. Now (tensile load × distance from centre of gravity to centre of pressure) + (weight of chimney ×

distance from axis of chimney to centre of pressure) balances the overturning moment due to wind. Thus, using inch ton units, $250t \times 22{\cdot}2 + 2 \times 12 = 500$, or $t = 476 \div 5{,}550 = 0{\cdot}086''$.

One ¾" B.S.W. bolt has a core area of 0·304 square inches, which allows bolts at a pitch of $\dfrac{0{\cdot}304}{0{\cdot}086}$ or 3½". The total circumference being approximately $32 \times \pi$, or 100", 32 bolts will be used.

Assuming a leverage from bolt line to root of angle of 1¼", the bending moment per inch on the angle is $0{\cdot}086 \times 5 \times 1{\cdot}25 = 0{\cdot}54$ inch tons. The Z of 1" of ⅝" thick material is $\dfrac{25}{64 \times 6} = 0{\cdot}065$ inch units, so that such a thickness of angle, with a stress of $0{\cdot}54 \div 0{\cdot}065$, or 8·3 tons per square inch, is suitable. A flat bar flange, say, 3" × ¾", would be equally satisfactory, if soundly welded to the chimney, whose circularity enables it to resist the bending effects.

Flanges higher up the chimney can be designed in a similar way and lighter sections could be used if thought advisable. In general the thickness of angle required varies as the distance down from the top of the chimney.

Alternatively, if the chimney is to stand on a concrete block, it will be better to assume that the lower 10′ 0″ or so is coned out to, say 4′ 0″ i/dia. and to design a base plate on the lines indicated in Example (1).

In such a case the extra bending moment is that resulting from a load of $1{\cdot}5 \times 5 \times 30 \div 2240$ or 0·1 ton at a leverage of 40 inches from the base plate, which gives an increase of 4 inch tons. Thus the total bending at the base plate is 504 inch tons for a wind load of $1{\cdot}67 + 0{\cdot}1$ or 1·77 tons, which means that the point of action of the wind is $504 \div 1{\cdot}77 = 285''$ or 23′ 9″ above the base plate. To allow for the cone and the base plate, the steel chimney can be assumed to weigh 2½ tons. (Alternative chimneys are shown on Fig. 5).

Concrete Block.

Assuming a concrete base block 4′ 0″ deep and D feet square in section, and a maximum pressure on the earth of 2 tons per square foot, a size of block can be determined by reference to Appendix 3.

Using first the conservative design as explained in part (*a*) of this appendix, and working on an overturning moment at the underside of the block of 1·77 × (23′ 9″ + 4′ 0″) or 49 foot tons, since W/A is approximately 4/16 or 0·25 tons per square foot for a block 4′ 0″ deep, $49 \div Z = 0{\cdot}25$. Thus $Z = 196$, or (see Z values in Appendix 3) $0{\cdot}118D^3 = 196$, whence $D^3 = 1660$, or D = say, 12′0″.

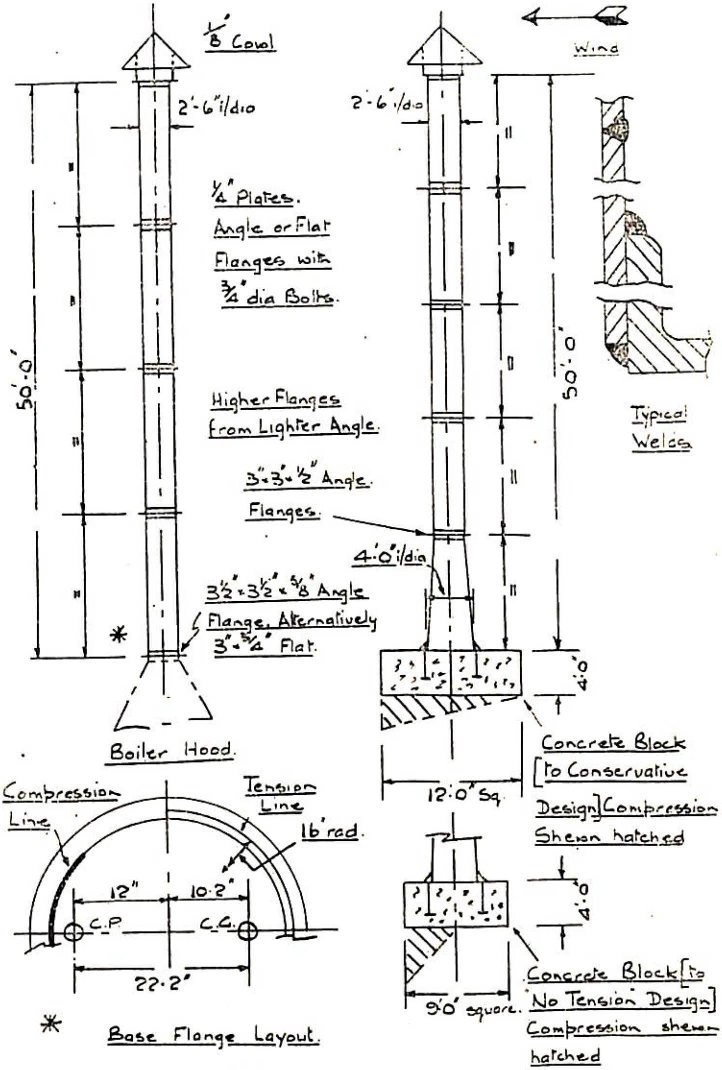

FIG. 5.

To check, the block weighs 4 × 144 ÷ 16 or 36 tons, so that the total weight of chimney and block is $2\frac{1}{2} + 36 = 38\frac{1}{2}$ tons. Thus W/A is $38\frac{1}{2} \div 144 = 0{\cdot}267$. Also $Z = 12^3 \times 0{\cdot}118 = 204$, which means that $M/Z = 49 \div 204 = 0{\cdot}24$. Hence W/A exceeds M/Z, and $W/A + M/Z = 0{\cdot}507$ tons per square foot, which is well within the prescribed limit.

If the "no tension" design as explained in part (*b*) of Appendix 3 is used, by the approximate method, $4 \times D^3 \div 48 = 49$ or $D^3 = 588$, whence D = say 8′ 0″.

To check, the block weighs $4 \times 8^2 \div 16 = 16$ tons, so that the total weight of chimney and block is $16 + 2\frac{1}{2}$ or $18\frac{1}{2}$ tons. Now the values of A and Z for half a square block are $D^2 \div 2$ and $D^3 \div 34$ respectively. Thus $W/A = 18{\cdot}5 \times 2 \div 8^2 = 0{\cdot}578$ tons per square foot. Also M is reduced by the weight of the block and chimney $\times\, e$, the eccentricity, that is by $18{\cdot}5 \times 8 \div 4{\cdot}24$ or 35 foot tons, so that $M/Z = (49 - 35) \div 8^3 \times 34 = 476 \div 512 = 0{\cdot}93$ tons per square foot compression on the leeward side. Thus $W/A + M/Z$ for the half block $= 0{\cdot}578 + 0{\cdot}93 = 1{\cdot}508$ tons per square foot, which again is well within the prescribed limit.[*]

A block 9′ 0″ square × 4′ 0″ deep is suggested as a final size.

APPENDIX 1—FITTINGS.

In the text and figures used to set out the principles of chimney design, little reference has been made to the various fittings required. Below are described the more common accessories, some of which are illustrated in Fig. 6.

(*a*) Top Curb.

Various types are used, ranging from M.S. flat or convex bar to elaborate cast iron copings. The purpose of the top curb is to stiffen the chimney, assist in the emission of smoke, and provide a finish and protection for any lining or coating. A mild steel angle is probably the best for all-round purposes.

(*b*) Ladder.

The larger chimneys usually have a ladder from roof or ground level to the top, and the two types generally fitted are :—(*a*) a normal rung ladder with a safety cage surrounding it ; and (*b*) a ladder consisting of 'U' shaped rungs tied together by a flat bar—the user to climb up inside the 'U' for safety. The pitch of the rungs is between 9″ and 12″, and intermediate platforms are sometimes fitted on the taller chimneys.

(*c*) Servicing Fittings.

As a rule, to assist painters, etc., pulleys, or a ring to which pulleys can be attached, are fixed near the top of the chimney so that working platforms can be raised and lowered as required.

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[*] It should be noted that M/Z on the windward side, *i.e.*, at the centre of the whole block, is $0{\cdot}93 \times 0{\cdot}5 = 0{\cdot}465$ tons per square foot tension.

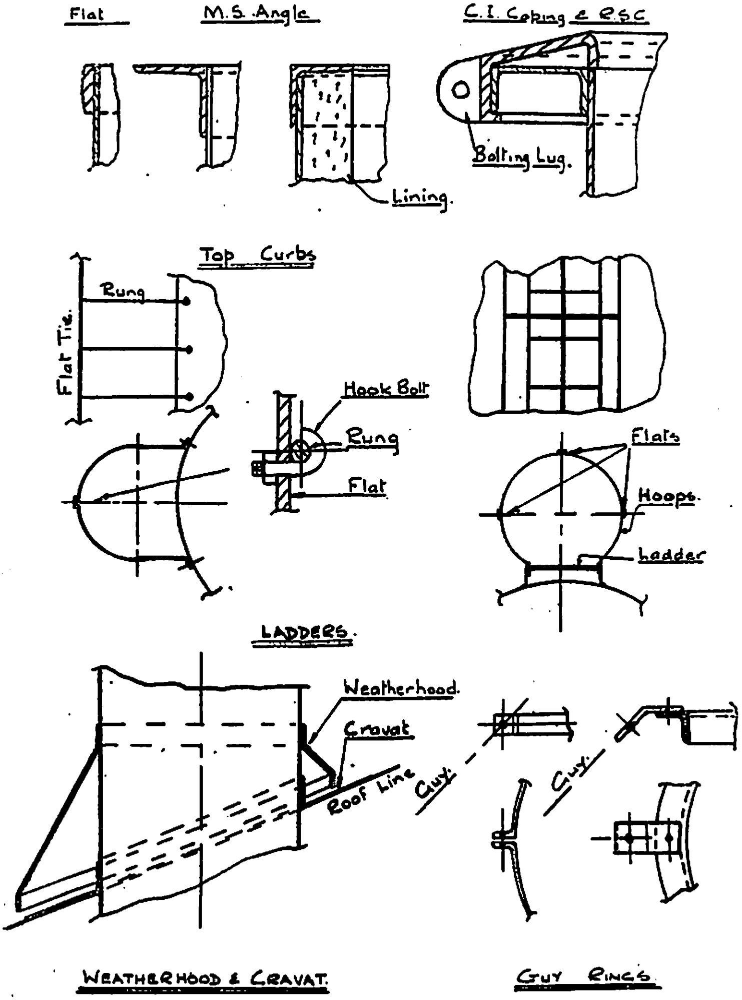

FIG. 6.

(*d*) **Weatherhood and Cravat.**

When a chimney passes through the roof of a building, a weatherhood, coned out and suitably shaped to match the roof slope, is fixed to the chimney ; and a cravat, or roof plate, is fitted around the corresponding hole in the roof. The cravat has a neck, slightly larger than the outside of the chimney, to exclude rain.

(*e*) **Guy Ring.**

On guyed chimneys, the wire guys are attached to a flat or angle ring, angle being preferable because it stiffens the chimney to withstand the concentrated loading at the guy level.

(*f*) **Access Door.**

To facilitate the inspection and lining of chimneys, and to enable soot, etc., to be removed, it is usual to fit an access door near the base. A common size on the larger chimneys is 3′ 0″ high × 2′ 0″ wide, the door frequently being hinged. As in the case of the flue opening in Example 1, any opening of this nature should be adequately compensated.

In addition to the above, some specifications call for one or more of the following :—inspection openings for sampling gases ; soot hopper, near the base and supplied with release valve ; division plate at flue inlet level, where two diametrically opposed flues enter ; lightning conductor, with point fixed to the top curb and copper tape cleated to the stack at intervals ; fan and venturi pipe to increase draught ; platforms at the top or other levels.

The flue gases enter either at the side of the chimney as in Example 1, or at the base via the foundation block. When chimneys are lined or coated, various kinds of cleats, shelf angles, etc., are required. Butterfly dampers are sometimes fitted in the smaller chimneys, but generally control of the flow of gases, together with the removal of grit, etc., takes place before the chimney is reached.

APPENDIX 2—VARIOUS FORMULAE, ETC.

Notation (units, tons and inches).

G = First moment of area about the centre line for half a section.

I = Second moment of area about the centre line for a full section.

P = Resultant horizontal guy force.

r = inside radius of a circle.

R = outside radius of a circle $= r + t$.

s = shear stress.

S = total shear force acting on a chimney.

t = thickness of plate.

y = distance to extreme fibre.

Z = modulus of section.

(A, D. H, K, p, T, w, x and z are also used with special meaning in several sections).

(*a*) Justification of Formula $Z = \pi r^2 t$ Approximately.

Adapting the standard formula $Z = I \div y$ to chimneys, the formula $Z = \frac{\pi}{4}\left(\frac{R^4 - r^4}{R}\right)$ or $\frac{\pi}{4}\left(\frac{(r+t)^4 - r^4}{r+t}\right)$ results. This becomes

$$Z = \frac{\pi}{4}\left(\frac{r^4 + 4r^3t + 6r^2t^2 + 4rt^3 + t^4 - r^4}{r+t}\right) \text{ or}$$

$$\frac{\pi t}{4}\left(\frac{4r^3 + 6r^2t + 4rt^2 + t^3}{r+t}\right)$$

By division

$$Z = \pi t\left(r^2 + \frac{rt}{2} + \frac{t^2}{2} - \frac{t^3}{4r+4t}\right)$$

Now, since t is usually a fraction, terms in t^3 and t^4 can be neglected, so that the true value of Z is very nearly $\pi t\ (r^2 + rt/2)$. If the approximate value of $\pi r^2 t$ is taken, the true value is very nearly $\frac{100 \times rt}{2r^2}$ or $\frac{100t}{2r}$ per cent. more than the approximate value. Normally this percentage is very small, and in any case the approximate value is on the safe side and so can be accepted. Furthermore, it is much easier to calculate than the more academic value.

The expression $Z = \pi r^2 t$ is also obtained if all the thickness t is assumed to be concentrated along the circle of radius r.

(*b*) Rivets in Vertical Seams to Resist Shear Forces.

If G and I are taken about the centre line of a section whose total thickness at the centre line is T, then the standard formula

$$s = \frac{S \times G}{I\,T} \text{ obtains.}$$

In the case of a chimney of inside radius r, when its thickness t is assumed to be concentrated along the inside circle, $I = \pi r^3 t$ and $T = 2t$. Also

$$G = 2\int_0^{\pi/2} r^2 t \cos A\,dA = 2r^2 t$$

$$\text{Thus } s = \frac{S \times 2r^2 t}{\pi r^3 t \times 2t} = \frac{S}{\pi r t} \text{ tons per square inch.}$$

Alternatively it can be said that the shear load per inch run along the vertical plate seam is st or $S \div \pi r$.

In the case of Example 1, the value of the wind force at the base, *i.e.*, S, is 10·6 tons, acting on a circle of diameter 10′ 6″.

Even if this force acted on the 7′ 0″ diameter section, the shear load per inch run of vertical seam would be $10{\cdot}6 \div (\pi \times 42) = 0{\cdot}08$ tons. Now a ¾″ rivet in bearing in ¼″ plate at 7·5 tons per square inch bearing stress can take a load of $\frac{13}{16} \times \frac{7{\cdot}5}{4} = 1{\cdot}5$ tons, so that theoretical considerations lead to the impractical pitch of $1{\cdot}5 \div 0{\cdot}08$, or 18¾″.

Thus it is sufficient to assume a nominal rivet pitch of 3″ – 4″ for vertical seams in order to resist shear forces. Further notes are given later in this Appendix.

(*c*) **Properties of a 45° Arc of a Thin Cylinder.**

Sectional area of arc $= \pi/8\ (R^2 - r^2) = 0{\cdot}39270\ (R + r)\ (R - r)$ (1)

First moment of area of arc about axis of cylinder $= 2 \int_r^R x^2 \sin 22\tfrac{1}{2}^\circ \, dx = 0{\cdot}25512\ (R^3 - r^3) = 0{\cdot}25512\ (R - r)\ (R^2 + Rr + r^2)$ (2)

The distance from the axis of the cylinder to the C.G. of the arc is (2) ÷ (1) $= 0{\cdot}64965 \left(\frac{R^2 + Rr + r^2}{R + r}\right)$ (3)

Also (1) × the square of (3) $= 0{\cdot}16574\ \frac{R - r}{R + r} \times (R^2 + Rr + r^2)^2$ (4)

†Second moment of area of arc about axis of cylinder $= \int_r^R \left(\frac{\pi}{8} + \frac{\sin 45^\circ}{2}\right) x^3 \, dx = 0{\cdot}18656\ (R^4 - r^4) = 0{\cdot}18656\ (R - r)\ (R + r)\ (R^2 + r^2)$ (5

Thus the second moment of area of the arc about its own C.G. is (5) – (4) $= \frac{R - r}{R + r} \{0{\cdot}18656\ (R + r)^2\ (R^2 + r^2) - 0{\cdot}16574\ (R^2 + Rr + r^2)^2\}$ (6)

Now $R = r + t$, and t^2 and higher powers of t can be ignored. Therefore (3) becomes $0{\cdot}64965 \times \frac{r^2 + 2rt + t^2 + r^2 + rt + r^2}{2r + t}$

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† See also calculations in Example (1), Flue Inlet.

$= \dfrac{1{\cdot}94895r\ (r+t)}{2r+t}$, and (6) becomes $\dfrac{t}{2r+t}\ \{0{\cdot}18656 \times 4r\ (r+t)\ 2r$ $(r+t)\ -\ 0{\cdot}16574\ (3r\ (r+t)\ )^2\}$

$$= \frac{r^2t\ (r+t)^2}{2r+t}\ [8 \times 0{\cdot}18656 - 9 \times 0{\cdot}16574]\ =\ \frac{0{\cdot}00082r^2t\ (r+t)^2}{2r+t}$$

Applying these formulae to the base of Example (1), where $r = 63''$ and $t = \frac{3}{8}''$, to determine the effect of two bolts each exerting a pull of 13·55 tons at $22\frac{1}{2}°$ centres on 10′ 11″ pitch circle diameter.

Total pull = 27·1 tons at $65\frac{1}{2} \times \cos 11\frac{1}{4}°$ or $64\frac{1}{4}''$ from centre of chimney. Approx. C.G. of arc (from (3)) $= \dfrac{1{\cdot}94895 \times 63 \times 63\frac{3}{8}}{126\frac{3}{8}}$ or 61·6″ from centre of chimney, therefore the bending moment = $27{\cdot}1 \times (64{\cdot}25 - 61{\cdot}6)$, or 72 ins. tons with respect to the arc.

The approx. second moment of area of the arc about its C.G. (from (6)) is, in inch units, $\dfrac{0{\cdot}00082 \times 63^2 \times \frac{3}{8} \times 63\frac{3}{8}^2}{126\frac{3}{8}} = 38{\cdot}75$, and y is either $(63{\cdot}375 - 61{\cdot}6) = 1{\cdot}775''$ to the outer extreme fibres, or $(61{\cdot}6 - 63 \cos 22\frac{1}{2}°) = 3{\cdot}4''$ to the inner extreme fibres. Thus Z for tension is 21·8 inch units and Z for compression 11·4.

Also the sectional area of the arc is $0{\cdot}39270 \times 126\frac{3}{8} \times \frac{3}{8} = 18{\cdot}5$ square inches. Therefore the direct tensile stress is $27{\cdot}1 \div 18{\cdot}5 = 1{\cdot}47$ tons per square inch, the tensile bending stress is $72 \div 21{\cdot}8 = 3{\cdot}3$ tons per square inch, and the compressive bending stress is $72 \div 11{\cdot}4 = 6{\cdot}3$ tons per square inch. Resultant values are :— tension, 4·77 tons per square inch ; compression, 4·83.

These values are only approximate, since the 45° arc is not usually isolated from the remainder of the circumference. Normally no calculation is required because the gussets stiffen the plate to withstand the locally high stress, which in this case is for 150% wind also.

(*d*) **Forces in Guy Ropes.**

In Fig. 7, diagram (*a*), 0 - 1 and 0 - 2 represent the horizontal components in two acting guy ropes (the third is redundant), and 0 - 3 represents the wind force—balanced by P—for an inclination to 0 - 1 of any angle $(120 - x)°$. In Fig. 7, diagram (*b*), A B C represents the triangle of forces for this condition. Now angle ABC is always 60°, force CA is equal in intensity to P, and angle ACB $= x°$.

Thus $P \div \sin 60^\circ = AB \div \sin x^\circ$, or $AB = \dfrac{P \sin x^\circ}{\sin 60^\circ}$, whose maximum value occurs when $x = 90^\circ$, and $120 - x = 30^\circ$. Hence the maximum horizontal component in any one guy is $P \times 1 \div \sin 60^\circ$ or $1{\cdot}15P$, and occurs when the wind is at 30° to that guy. The horizontal component in the other acting guy is $1{\cdot}15P \div 2$ for this condition.

Also from the above diagrams $AB = \dfrac{P \sin x^\circ}{\sin 60^\circ}$ and $BC = \dfrac{P \sin (120 - x)^\circ}{\sin 60^\circ}$. Thus $AB + BC = \dfrac{P}{\sin 60^\circ} (\sin x^\circ + \sin (120 - x)^\circ)$, which is a maximum when $\sin x^\circ + \sin (120 - x)^\circ$ is a maximum.

Now $\sin x^\circ + \sin (120 - x)^\circ = \sin x^\circ + \sin 120^\circ \cos x^\circ - \cos 120^\circ \sin x^\circ = \sin x^\circ + \sqrt{3}/2 \cos x^\circ + \tfrac{1}{2} \sin x^\circ = 3/2 \sin x^\circ + \sqrt{3}/2 \cos x^\circ$. Differentiating and equating to zero,

$$3/2 \cos x^\circ - \sqrt{3}/2 \sin x^\circ = 0, \text{ whence } \sqrt{3} = \tan x^\circ, \text{ or } x = 60^\circ.$$

This means that $AB + BC = \dfrac{P}{\sin 60^\circ} (\sin 60^\circ + \sin (120^\circ - 60^\circ) = 2P$ so that the maximum sum of the horizontal components in any two acting guys is 2P, each horizontal component being equal.

From Fig. 7, diagram (*c*), it is clear that the vertical component in a guy rope is tan A × the horizontal component, and that the resultant force in a guy rope is sec A × the horizontal component. Thus the maximum values to be allowed for are :—resultant force in any one guy, $\sec A \times 1{\cdot}15P$; vertical component in any one guy, $\tan A \times 1{\cdot}15P$; sum of vertical components in any two guys, $\tan A \times 2P$. (A is the angle of inclination of the guy rope to the horizontal).

(*e*) **Development of Chimney Plates.**

In the case of riveted chimneys, all the plates form part of cones. The taper of the cones in the parallel portion of the chimney is very small, while that in the coned-out portion of the chimney is appreciable. The dimensions of the developed plates are calculated or set out from the dimensions at the mean thickness of the plates when rolled or set to the final conical shape.

In Fig. 7, diagram (*d*) shows, with exaggerated thickness, the section of a typical plate in the parallel portion of a chimney. If the seam pitch is 60″, the plate thickness ¼″, and the mean radius 42¼″, then the mean line of the plate makes with the vertical an angle A whose sine is $\frac{t}{2} \div \frac{\text{pitch of seams}}{2} = \frac{1}{4 \times 2} \div \frac{60}{2}$ $= \frac{1}{240}$. But this angle is also the semi-vertical angle of the full mean line cone of mean radius $42\frac{1}{4}''$, so that the slant height of the full mean line cone is $42\frac{1}{4} \times 240$, or 10,140″. Thus the vertical angle of the full cone when developed is $\frac{360 \times 42\frac{1}{4}}{10{,}140} = \frac{360 \times 42\frac{1}{4}}{42\frac{1}{4} \times 240}$ $= 1\frac{1}{2}\%$. If two plates per strake are used, the vertical angle of each developed plate is $\frac{3}{4}°$, as shown in Fig. 7, diagram (*e*), where the chain dot lines indicate rivet lines, and the full lines the edges of the plate. The line of mean circumference has a total length of mean radius, *i.e.*, $42\frac{1}{4}''$, multiplied by π for each half strake.

It will be appreciated that the radius of 10,140″ cannot be set out on a template shop floor, and that $\frac{3}{4}°$ is hardly measurable by ordinary means. Thus some form of approximation is necessary to draw the developed shape of each half strake. Fig. 7, diagram (*f*), shows a circle of radius R with a chord of length $2x$, drawn at right angles to a diameter, and dividing this diameter into two lengths z and $2R - z$. Now $x^2 = z\ (2R - z) = 2Rz - z^2$ (as proved in any standard book on geometry). If z is very small, little error is involved when x^2 is taken to equal $2Rz$, whence $z = \frac{x^2}{2R}$. In other words, the circumference of a very large circle can be assumed to be parabolic in shape for small arcs.

Since sin A is $\frac{1}{240}$, it can be assumed that the lower and upper curved rivet lines in Fig. 7, diagram (*e*), have radii of 10,170″ and 10,110″ respectively. If a line of length, say 100″, is drawn perpendicular to the centre line of the developed half strake, the value of z as denoted in Fig. 7, diagram (*f*), is $\frac{100^2}{2 \times 10{,}170}$ or $\frac{10{,}000}{20{,}340}$ $= 0{\cdot}492''$, say $\frac{1}{2}''$, for the lower curved line. Corresponding values of $\frac{9}{32}''$, $\frac{1}{8}''$, and $\frac{1}{32}''$, can be set out for z for distances of 75″, 50″, and 25″ respectively on either side of the centre line of the developed half strake, and a curve drawn through the points so determined. If a length of $42\frac{1}{4} \times \pi \times \frac{10{,}170}{10{,}140} \div 2$ or $66{\cdot}56''$ is set out on either side of the centre line along this curve, then the required lower curved length is determined.

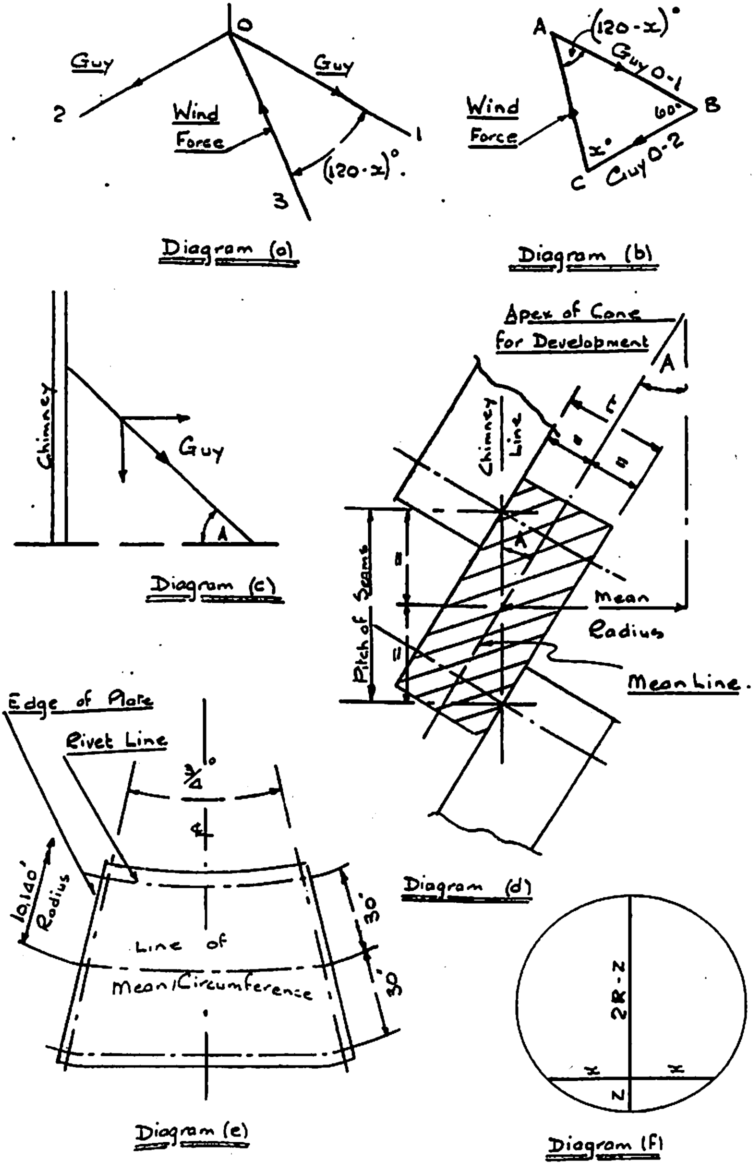

Diagram (a)

Diagram (b)

Diagram (c)

Diagram (d)

Diagram (e)

Diagram (f)

FIG 7.

The upper curved line can be drawn similarly, and both curves can be divided into a suitable number of equal parts to give the required rivet pitch. Thus if 4″ pitch is required, 34 equal parts will be necessary over the lower total curved length of 133·12″, and the same number over the upper curved line.

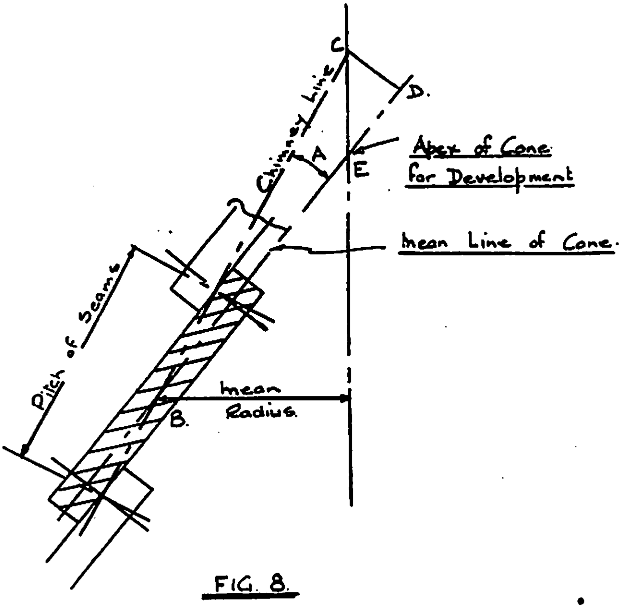

FIG. 8.

With respect to the development of plates forming the coned-out portion of a chimney, Fig. 8 shows a typical plate in section. If the mean radius is 72″ and the taper of the chimney line cone is 12″ in 20′ 0″, the vertical height of this cone is $\frac{72}{12} \times 20 = 120' \, 0''$. Thus the slant height BC of the chimney line is $\sqrt{120^2+6^2} = 120 \cdot 15'$. If the pitch of seams is 60″, and the plate thickness $\frac{3}{8}''$, sin A $= \frac{1}{160}$, reasoning as above. This means that in Fig. 8, dimension CD $= 120 \cdot 15' \times \frac{1}{160}$. Now since A is a very small angle, BC and BD are practically equal, and angle CED can be taken as equal to angle BCE, the semi-vertical angle of the chimney line cone, whose cotangent is $\frac{120}{6}$. ED = CD cot CED = CD cot BCE

$= \dfrac{120{\cdot}15}{160} \times \dfrac{120}{6} = 15{\cdot}02'$. Thus BE the slant height of the full mean line cone is $120{\cdot}15' - 15{\cdot}02' = 105{\cdot}13'$, and the vertical angle of the full developed cone is $\dfrac{360 \times 6}{105{\cdot}13} = 20{\cdot}55^\circ$. At three plates, per strake, each developed plate has a vertical angle of $6{\cdot}85^\circ$, and a mean circumference line of 105·13′ radius. Dimensions can be worked out as for the plates in the parallel portion of the chimney.

It will be seen that the $\frac{1}{16}''$ rivet hole clearance is necessary to allow for unavoidable errors in templating, and that much is left to the discretion of the template maker where actual rivet pitching is concerned. The welded chimney, where the seams butt together, avoids difficult templating for the parallel portion, but great care is necessary in developing the plates for the coned-out portion to ensure that curves set out as indicated above fit together accurately at site.

(*f*) **M/Z Stress at Beginning of Coned-out Portion. (See Fig. 9).**

Taking a chimney with parallel height and diameter H and D feet respectively, an increase in diameter of the chimney over the coned-out portion of K ft. per ft. of height, a wind force of w tons per square foot, and a constant thickness of t'', the M/Z stress at x ft. down the coned-out portion (in tons per square inch) is :—

$$\frac{12\, w \left[D \dfrac{(H+x)^2}{2} + \dfrac{Kx^3}{6} \right]}{\dfrac{\pi\ (D+Kx)^2 \times t \times 144}{4}} \quad \text{or} \quad \frac{2\, w \left[3D\ (H+x)^2 + Kx^3 \right]}{\dfrac{\pi \times t \times 144}{4}\ (D+Kx)^2}$$

If the constants are left out, this becomes $\dfrac{3D\ (H+x)^2 + Kx^3}{(D+Kx)^2}$

Differentiating and leaving out the denominator, which must be positive because it is a square, the expression gives

$$(D+Kx)^2\ (3D\ (2H+2x) + 3Kx^2) - 2K\ (D+Kx)\ (3D\ (H+x)^2 + Kx^3),$$

or $(D+Kx)\ (3D\ (2H+2x) + 3Kx^2) - 2K\ (3D\ (H+x)^2 + Kx^3)$,

cancelling $D + Kx$.

If $x = 0$, *i.e.*, where the cone begins, this last expression reduces to $6\ D^2H - 6\ KDH^2$, which must be negative if the M/Z stress is to begin to decrease where the coned-out portion commences.

This means that 6 KDH^2 must exceed 6 D^2H,
that KH ,, ,, D,
or that K ,, ,, D/H.

In a chimney where the height of the coned-out portion is $H/4$, the base diameter should at the least be $D + D/H \times H/4$ or $1\frac{1}{4}$ D. Since in such a case the base diameter would normally be $1\frac{1}{2}$ D, the M/Z stress would show an appreciable decrease at the beginning of the coned-out portion—as mentioned in Example 2.

(*g*) **Local Bending Due to Wind.**

While to determine the M/Z stress the wind pressure is assumed to act uniformly over the projected area, experiments show that the distribution of wind pressure around the circumference of a chimney is rather complex. In general it is found that pressure is exerted radially inwards over about half of the windward face and suction radially outwards over the remainder of the circumference, or, in other words, the majority of the wind force is suction on the sides and rear of the chimney. For practical purposes it is probably sufficient to assume the following distribution, where p is the wind pressure in lbs. per square foot (as derived from, say, B.S. 449 without allowing for the rotundity factory of 0·7) and positive and negative signs mean pressure inwards and suction outwards respectively :—

(1) 45° windward arcs either side of the wind direction, $+ P/2$ acting radially.
(2) 45° leeward arcs either side of the wind direction, $- P/2$ acting radially.
(3) Remaining 180°, $-p$ acting radially.

It will be observed that the forces on (1) and (2) give, for a diameter D feet, approximately $0{\cdot}7\, p\, D$ total force per foot of height in the direction of the wind, and that, owing to the symmetrical distribution, all pressures or suctions at right angles to the wind are balanced.

Calculations based on the above assumptions show that there is a maximum bending moment outwards of approximately $0{\cdot}06\, p\, D^2$ ft. lbs. per foot height in the chimney plates and vertical seams. Or, making some allowance, 30 per cent. say, for more concentration of pressure or suction than assumed above, it may be concluded that there is a maximum bending moment of $\frac{p\, D^2}{2400}$ inch tons per foot height tending to distort the chimney into an oval shape.

Now both plates and rivets should be suitable for this bending moment. Referring to design example number 1, $p = 32{\cdot}7$,

$D = 7$, and the upper plate thickness is $\frac{1}{4}''$. Thus the bending moment to be considered is $32{\cdot}7 \times 49 \div 2400 = 0{\cdot}67$ ins. tons per foot. The Z of 12″ of $\frac{1}{4}''$ plate is $\frac{12}{16 \times 6} = \frac{1}{8}$, therefore the bending stress is $8 \times 0{\cdot}67 = 5{\cdot}36$ tons per square inch, which is acceptable.

Also, assuming $\frac{3}{4}''$ diameter rivets, single row, $1\frac{1}{4}''$ edge distance, and working on a rather low tensile stress of 2 tons per square inch over the $\frac{13}{16}''$ driven diameter of rivet, each rivet exerts a force of 1 ton approx. at a leverage of roughly $\frac{3}{4}''$—this being the distance from rivet to assumed centre of compression between the two plates riveted together. In other words each rivet can safely resist a bending moment of $\frac{3}{4}$ inch tons, or 1 rivet per foot of height is sufficient in the vertical seams to resist bending in this instance.

Another way of determining the maximum rivet pitch in vertical seams is to make the riveting as strong as the plate. Thus the Z of 12″ of $\frac{5}{16}''$ plate is $\frac{12 \times 25}{6 \times 256} = \frac{1}{5}$ approx., so that with a stress of 10 tons per square inch the plate can resist a bending moment of 2 ins. tons per foot. Therefore, the number of rivets per foot should be $2 \div \frac{3}{4} = \frac{8}{3}$, *i.e.*, $4\frac{1}{2}''$ pitch.

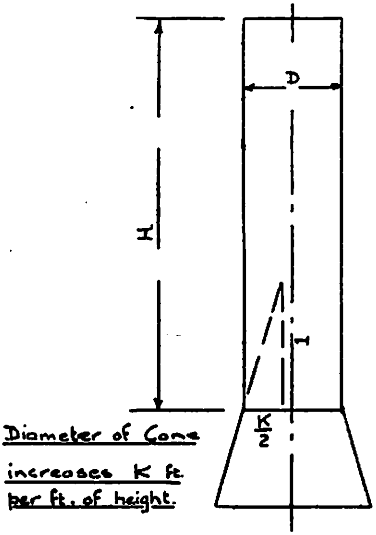

FIG 9.

(*h*) **Alternative Method of Base Design.**

If the base is assumed to be a circle D feet diameter × T inches wide over 240° of the circumference and 5·6 T inches wide over the remaining 120°, then it forms a shape with the following properties :—

C.G. at $D \div 4$ feet from the chimney centre.
Area $= 96\,DT$ ins.^{2}.
Min. $Z = 144\,D^2T$ ins.^{3}.
Ratio of maximum Z to minimum Z = 3 to 1.

Allowing for the fact that the weight of the chimney is $D \div 4$ feet eccentric about the C.G. of the above figure, it is possible to determine $\frac{M}{Z}$ and $\frac{W}{A}$ in terms of T, $\frac{M}{Z}$ being calculated for minimum and maximum values of Z.

For the bolts $\frac{M}{Z} - \frac{W}{A}$ (using minimum Z) is equated to, say, 5 tons per square inch, from which T is found. The core area of each bolt is T × bolt pitch in inches. For the width of base plate T is multiplied by 5·6 and 15—the ratio of E for steel and concrete—or width of baseplate = 84 T inches. The maximum steel to concrete stress is $\frac{M}{Z} + \frac{W}{A}$ (using maximum Z) divided by 15.

Referring to the first design example, the chimney weighs 23·5 tons, the base diameter is 10′ 6″ and the overturning moment is 1060 ft. tons.

Thus nett O.T.M. $= 1060 - 23{\cdot}5 \times 2{\cdot}625 = 1000$ ft. tons approx.

$$\frac{M}{Z} \text{ using minimum } Z = \frac{1000 \times 12}{144 \times 10{\cdot}5^2 \times T} = 0{\cdot}76 \div T \text{ tons per sq. ins.}$$

$$\frac{M}{Z} \text{ using maximum } Z = 0{\cdot}253 \div T \text{ tons per sq. ins.}$$

$$\frac{W}{A} = \frac{23{\cdot}5}{96 \times 10{\cdot}5\,T} = 0{\cdot}023 \div T \text{ tons per sq. ins.}$$

From these values, $0{\cdot}737 \div T = 5$, or $T = 0{\cdot}147$ inches. Therefore, assuming 16 equal pitches of 25″, the core area of each bolt = 3·7 square inches, for which 2½″ B.S.W. will suffice. That is 16 – 2½″ dia. bolts are required.

Also the base plate is $84 \times 0{\cdot}147 = 12\tfrac{1}{2}$″ wide, and the maximum steel to concrete stress is $\frac{0{\cdot}276 \times 144}{0{\cdot}147 \times 15} = 18$ tons per square foot, which is acceptable. The thickness of the base plate, the gussets, and the base angle can be determined as shown earlier. The assumed stress of 5 tons per square inch can be reduced if the steel to concrete stress is considered too high.

APPENDIX 3—CONCRETE FOUNDATION BLOCKS.

Notation :—

W_c = weight of concrete block, in tons (at 140 lbs. per cubic foot for concrete, 16 cubic feet equals 1 ton).
W_s = weight of steel chimney (plus lining, if any) in tons.
W = total weight of block and chimney, in tons $= W_c + W_s$.
A = area of underside of block in effective contact with the earth, in square feet.
M = overturning moment at underside of block, in foot tons.
Z = modulus of section of block in effective contact with the earth, in foot units. (The minimum value of Z is used).
H = height of block, in feet.
D = diameter, or distance over flats, of block, in feet.

(*a*) Conservative Design.

For this design the whole of the underside of the block is assumed to be in effective contact with the earth, and the following relations between $\frac{W}{A}$ and $\frac{M}{Z}$ obtain :—

$\frac{W}{A} + \frac{M}{Z}$ does not exceed the maximum stress allowed for the soil in question; and $\frac{W}{A}$ equals or exceeds $\frac{M}{Z}$. Also $\frac{W}{A} = \frac{H}{16}$ tons per square foot, when the chimney weight is neglected.

In Fig. 10*a*, where X – X is the underside of the block, diagrams 1 and 2 show separately the $\frac{W}{A}$ and $\frac{M}{Z}$ stresses, and diagram 3 shows their combined effect for the ideal base.

The first two bases calculated for design example 1, and that shown on Fig. 2, have been worked out on the above principles. The sizes obtained are on the large side. Below is given a list of properties for use in designing blocks by this first method, A and Z being for the whole sections of the block.

Shape	A	Z (min.)	W_c
Square	D^2	$0{\cdot}118\ D^3$	$HD^2 \div 16$
Hexagon	$0{\cdot}866\ D^2$	$0{\cdot}104\ D^3$	$HD^2 \div 18{\cdot}5$
Octagon	$0{\cdot}828\ D^2$	$0{\cdot}101\ D^3$	$HD^2 \div 19{\cdot}3$
Circle	$0{\cdot}7854\ D^2$	$0{\cdot}098\ D^3$	$HD^2 \div 20{\cdot}4$

Base Block Properties for (a) Conservative Design

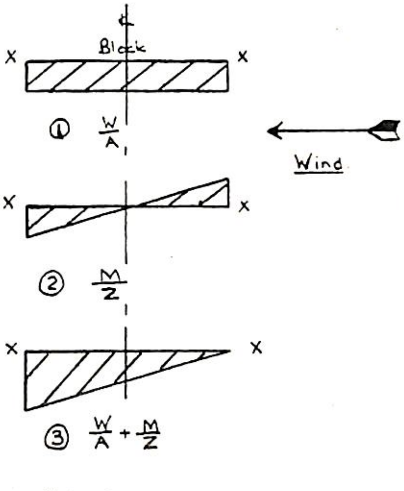

FIG. 10a.

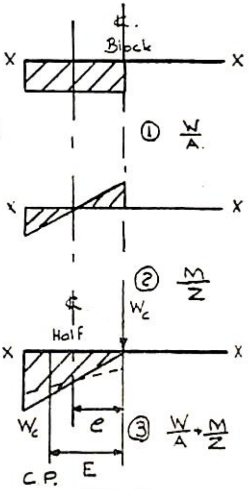

FIG. 10b.

(*b*) **"No Tension" Design.**

For this design it is assumed that a portion of the windward side of the block is not in effective contact with the earth, the portion being left to the discretion of the designer. In this instance, it is assumed that the windward half of the block is not in effective contact, and it is suggested that the block finally used should lie somewhere between those obtained by design methods (*a*) and (*b*).

Referring to Fig. 10*b*, diagram 3 shows the resultant triangular stress diagram assumed. The centre of pressure, or C.P., is at a distance E from the centre of the whole block, the underside of which is represented by line X – X. Ignoring the value of W_s, the concrete block exerts a restraining moment of $W_c \times E$, which equals the overturning moment M. Now W_c and E can be expressed in terms of H and D, so that, for any given value of H, D can be obtained.

However, when W_s is added, the restraining moment becomes $(W_c + W_s) \times E$, which is somewhat greater than $W_c \times E$, or M, and the block already calculated is slightly larger than necessary. This is no detriment, because it makes for extra stability. Furthermore, it is possible with experience to increase the product $W_c \times E$ by a percentage, say 10%, to allow for the quantity W_s, and so arrive at a more accurate base size.

In the table below values are given for $W_c \times E$, W_c, and a quantity K by which the value $W_c + W_s$ should be divided to give the maximum stress on the earth. The value of H should not exceed 6′ 0″ for a maximum earth pressure of 2 tons per square foot.

Shape	$W_c \times E$	W_c	K
Square	$HD^3 \div 48$	$HD^2 \div 16$	$D^2 \div 6$
Hexagon	$HD^3 \div 60$	$HD^2 \div 18{\cdot}5$	$D^2 \div 6$
Octagon	$HD^3 \div 63{\cdot}5$	$HD^2 \div 19{\cdot}3$	$D^2 \div 6$
Circle	$HD^3 \div 69{\cdot}2$	$HD^2 \div 20{\cdot}4$	$D^2 \div 6$

Base Block Properties for (b) "No Tension" Design (Approximate Method Only)

Alternatively the block so calculated can be checked by the $\frac{W}{A}$ and $\frac{M}{Z}$ values for the leeward half only. It should be noted that, since the centre line, or neutral axis, of this half is eccentric by an amount e (see Fig. 10*b*, diagram 3) with respect to the centre line of the whole block, the value of M should be reduced by the product $(W_c + W_s) \times e$. Tabulated below are values of A and Z (for the leeward half), e and W_c. The maximum stresses obtained are rather less than those derived by the use of ratio K above. This arises because the resultant of the $\frac{W}{A}$ and $\frac{M}{Z}$ stresses—shown separately in Fig. 10*b*, diagrams 1 and 2—is usually a trapezium shaped figure (indicated by dotted line in Fig. 10*b*, diagram 3) instead of a triangular one. In other words $\frac{W}{A}$ exceeds $\frac{M}{Z}$. The value of H should not exceed 6′ 6″ for a maximum earth pressure of 2 tons per square foot. Blocks are calculated by the "no tension" method in design examples 1 and 4.

The effect of the earth on the sides of concrete blocks has been neglected because of uncertain working stresses and the possibility of nearby excavations, etc.

Shape	A	Z (min.)	e	W_c
Square	$D^2 \div 2$	$D^3 \div 34$	$D \div 4{\cdot}24$	$HD^2 \div 16$
Hexagon	$D^2 \div 2{\cdot}31$	$D^3 \div 41{\cdot}5$	$D \div 4{\cdot}50$	$HD^2 \div 18{\cdot}5$
Octagon	$D^2 \div 2{\cdot}41$	$D^3 \div 41{\cdot}5$	$D \div 4{\cdot}60$	$HD^2 \div 19{\cdot}3$
Circle	$D^2 \div 2{\cdot}55$	$D^3 \div 42$	$D \div 4{\cdot}72$	$HD^2 \div 20{\cdot}4$

Base Block Properties for (b) "No Tension" Design

$\left(\frac{W}{A} + \frac{M}{Z}\right.$ **Method** $\left.\right)$

Note.—The M/Z stresses obtained by the use of the minimum Z values given in this table are for compression at the leeward edge of the block. If it is desired to find the tensile M/Z stress at the centre line of the whole block, the compressive M/Z stress should be multiplied by the following factors :—

Square, 0·50 ; Hexagon, 0·625 ; Octagon, 0·675 ; Circle, 0·735. This difference is due to the fact that the half sections are not symmetrical about their neutral axes.

APPENDIX 4—BENDING MOMENTS AND REINFORCEMENT IN FOUNDATION BLOCKS.

Tables 5 and 6 respectively show the bending moments in foot tons in foundation blocks at various distances from the centre for uniform loading of 1 ton per square foot, and for loading varying uniformly from 1 ton per square foot at the edge to zero at the centre. Curves (1) are for a square of 10′ 0″ side when bending occurs along a line parallel to a side, curves (2) are for the same square when bending occurs along a line parallel to a diagonal, and curves (3) are for a circle of 10′ 0″ diameter. For other loadings and sizes, where D is the length in feet of the side of a square or the diameter in feet of a circle, the plotted values should be multiplied by the product of the uniform or edge stress, in tons per square foot, and $D^3 \div 1000$. Curves (3) apply approximately to regular polygons when D is the average of their inscribed and circumscribed circles.

Fig. 10*c* shows hatched the width of foundation block assumed to be effective in resisting bending moments as obtained from Tables 5 and 6, the loading from the areas not hatched being transferred to the hatched area by means of the reinforcement at right angles to that in the effective area. The C.P., or centre of pressure, is at the point assumed in the design of the chimney base plate, and the C.G., or centre of gravity, of the holding down bolt pull can be assumed to be at a distance of P.C.D. $\div \pi$ from the centre of the block. Some dispersal of loading about the C.P. and C.G. should be allowed for.

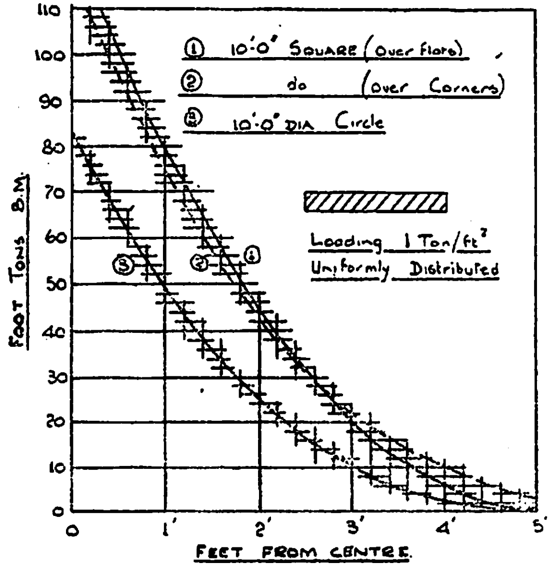

TABLE 5.

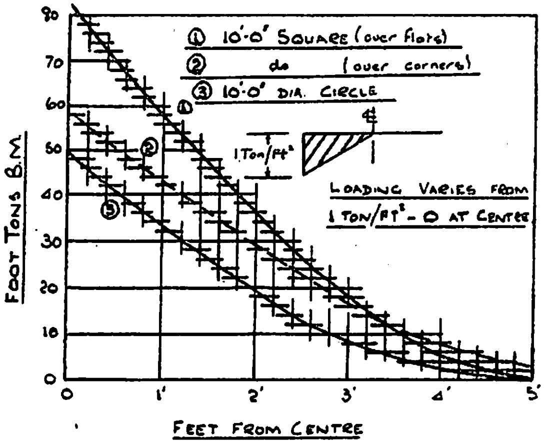

TABLE 6.

If it is desired to calculate the bending moment for the 6′ 0″ deep × 32′ 0″ diameter block for Example 1, the method below is suggested.

Weight of steel chimney = 23·5 tons.
Area of base = 804 sq. ft.
Overturning moment at underside of base = 1160 foot tons.
Z of base = 3216 ft. units.

Thus nett $\frac{W}{A}$ uniform stress $= \frac{23{\cdot}5}{804} = 0{\cdot}0292$ tons per square foot. (The weight of the concrete block cancels out the W/A stress resulting from the block). Also the maximum edge stress, or $\frac{M}{Z}$, is $\frac{1160}{3216} = 0{\cdot}36$ tons per square foot. Therefore the curve (3) values plotted in Tables 5 and 6 respectively must be multiplied by $\frac{0{\cdot}0292 \times 32^3}{1000}$ and $\frac{0{\cdot}36 \times 32^3}{1000}$ or by 0·95 and 11·8, the distances from the centre as plotted being multiplied by 3·2 to suit the 32′ 0″ diameter. The M/Z stresses on the leeward and windward halves respectively cause bending upwards and downwards in the foundation block.

The C.P. is at 4′ 9″ from the centre, and the compression load is 146 tons. The C.G. is at $11 \times \frac{7}{22}$ or 3′ 6″ from the centre, the total tension being 122 tons. Distances 4·75′ and 3·5′ read 1·5′ and 1·1′ respectively on the 10′ 0″ diameter graph.

Below are set out the bending moments at several positions of the base block, the negative sign being for bending upwards, and for distances leeward from the centre. (The W/A stress causes bending upwards).

Distance	−6·4′	−4·75′	−3·2′	Centre	+3·2′	+3·5′	+6·4′
Table 5	−24	−34	−47	−80	−47	−44	−24
Table 6	−224	−307	−390	±580	+390	+380	+224
Compression	0	0	+226	+695	—	—	—
Tension	—	—	—	−426	−37	0	0
Resultant	−248	−341	−211	+35 or +74 (Calc. errors)	+306	+336	+200

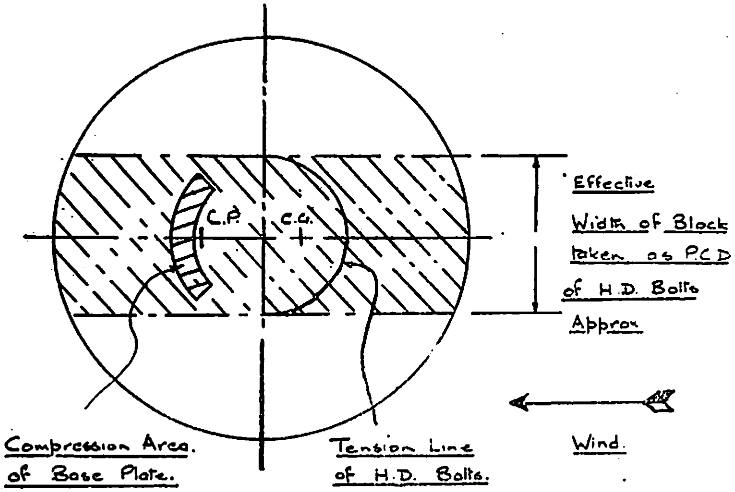
C.P.
C.G.
Effective Width of Block taken as P.C.D. of H.D. Bolts Approx.
Wind.
Compression Area of Base Plate.
Tension Line of H.D. Bolts.

FIG. 10c.

These values are shown on Fig. 11, the peak values being rounded off to show the effects of loading dispersal. The maximum values to work on are taken as ± 310 ft. tons over an effective width of 11′ 0″, or, say, 28 ft. tons per foot width.

Assuming an effective concrete depth of 56″, of which 10″ are in compression, and ignoring any steel in the compression area of the concrete, the leverage of steel or concrete is 46 + 6⅝ or 52⅝″, which gives a load on steel or concrete of 28 × 12 ÷ 52⅝ = 6·4 tons or 14,400 lbs. Thus the concrete stress is 14,400 ÷ 12 × 10/2 = 240 lbs. per square ins., and the steel stress 240/10 × 46 × 15 or 16,500 lbs. per square ins. The concrete stress is suitable, and the area of steel required is 14,400 ÷ 16,500, or 0·88 square inches per foot, *i.e.*, ¾″ dia. bars at 6″ pitch.

This amount of steel should be fixed in two perpendicular directions to provide for varying directions of wind and to give distribution of loading. Both top and bottom of the block require such reinforcement.

Since the maximum compressive loading on the block is only 0·0292 + 0·36, or 0·39 tons per square foot, the bending moment allowed for above is not likely to be exceeded, although a check might be advisable for strips at right angles to the wind direction.

The permitted punching load on the block at 150 lbs. per square inch stress is approx. 200 × 56 × 150 ÷ 2240 = 750 tons, assuming that the baseplate over the compressive area has a perimeter of some 16′ 8″; and the permitted shear load over the 11′ 0″ width, at 75 lbs. per square stress, is $11 \times 12 \times 56 \times 75 \div 2240$ or 247 tons. Neither of these values is exceeded, so that the block is suitable for the loads imposed.

As a further example in the use of Tables 5 and 6, the 4′ 0″ deep × 8′ 0″ square block designed on the "no tension" method for example 4 will be taken. The data already calculated are :—

Weight of concrete, 0·25 tons per square foot of area ; $\frac{W}{A} + \frac{M}{Z}$ at leeward edge, 1·508 tons per square foot ; $\frac{W}{A} - \frac{M}{Z}$ stress at centre of block, 0·578 − 0·465 or 0·113 tons per square foot.

Resolving the stresses, the $\frac{W}{A}$ and $\frac{M}{Z}$ values are equivalent to a uniform upward loading of 0·113 tons per square foot over the leeward half, together with an upward loading ranging from (1·508 − 0·113) or 1·395 tons per square foot at the edge to zero at the centre over the leeward half. The self-weight of the concrete imposes a downward loading over the whole area of 0·25 tons per square foot, which means a nett downward loading of (0·25 − 0·113) or 0·137 tons per square foot over the leeward half. All these are for wind parallel to a diagonal.

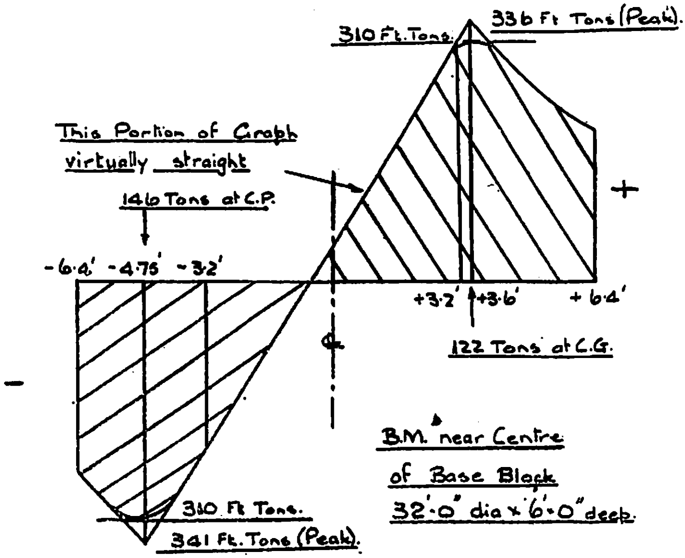

FIG. 11.

Thus the values plotted for Table 5, curve 2, must be multiplied by the following factors :—

Leeward half $0{\cdot}137 \times \frac{8^3}{1000}$ for B.M. (8/10 for distances).

Windward half $0{\cdot}25 \times \frac{8^3}{1000}$,, ,, (,, ,, ,,).

(These two bending moments act downwards).

Likewise the values plotted for Table 6, curve 2, must be multiplied by the factor $1{\cdot}395 \times \frac{8^3}{1000}$ for B.M. (acting upwards over the leeward half only), and 8/10 for distance. The bending moments resulting from the chimney baseplate and holding down bolts have also to be allowed for, the final result being on the lines of Fig. 11.

In the case of a square block a check should be made for the wind parallel to the sides, it being remembered that the values of e and Z (see table for base block properties, "no tension" design, $\frac{W}{A} + \frac{M}{Z}$ method, in Appendix 3) are $\frac{D}{4}$ and $\frac{D^3}{24}$ respectively, and that the shape $D \times D/2$ is symmetrical about its neutral axis, *i.e.*, that the compressive and tensile M/Z stresses are equal in value.

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